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17 Abstract

The joint Canada/Japan Paving in Cold Areas Mini Workshop held in Vancouver between October 18 - 22, 1982 brought together Canadian and Japanese experts in the areas of pavement research to discuss the particular problems of paving in cold areas. The workshop consisted of six sessions at which twenty-three papers were presented and one panel discussion took place. The six major topic areas covered in the workshop were: low temperature cracking, materials and construction to overcome freezing problems in soils, frost penetration and the prevention of frost heaving, the prevention and cure of reflection cracking, characteristics of paving materials, and additives in asphalts.

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17 Résumé

Le mini-atelier conjoint Canada-Japon intitulé Les revêtements dans les climats froids tenu à Vancouver entre les 18 et 22 octobre 1982 a réuni un groupe d'experts canadiens et japonais dans le domaine de la recherche sur les revêtements afin de discuter les problèmes particuliers des revêtements dans les climats froids. L'atelier a compris six séances, durant lesquelles on a présenté vingt-trois communications techniques, et une session de discussion par des panelistes. Les six sujets généraux discutés pendant l'atelier furent: la fissuration à basse température, les matériaux de construction utilisés afin de combattre les problèmes de gel dans les sols, la pénétration du gel et les mesures préventives contre le gonflement dû au gel, les mesures préventives contre la fissuration due à la réflexion, les caractéristiques des matériaux utilisés pour les revêtements, et les additifs utilisés dans les asphaltes.

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FOREWORD

At the 4th Canada/Japan Consultations on Science and Technology held in Tokyo, June 3-6, 1980, it was agreed that one of the topics where information exchange and collaborative activity would be of mutual benefit is Paving in Cold Areas.

The contact points named to foster this activity are:-

William A. Phang Head, Pavement Research Ontario Ministry of Transportation and Communications	and	Takashi Iijima Head, Pavement Division Public Works Research Institute Japan Ministry of Construction

Information exchange was initiated during a visit to Japan by W.A. Phang and I.F. Gilbert in November 1980, and was continued during reciprocal visits to Canada by T. Sugawara, S. Matsuno, H. Kubo, and H. Ishii at various times in 1981. This Mini-Workshop is intended to further extend the depth and scope of the exchange by involving larger numbers of Canadian and Japanese engineers.

The first announcement and call for papers for the Mini-Workshop was issued in November 1981. The response from both Canadian and Japanese engineers is most gratifying.

The papers presented in the Proceedings are organized in the same way as in the program and consist of the following:-

Session I -Low Temperature Cracking
Session II -Materials and Construction to Overcome Problems of Freezing in Soils
Session III-Frost Penetration and Prevention of Frost Heave in Pavements
Session IV -Reflection Cracking, Prevention and Cure
Session V -Paving Materials
Session VI -Additive Research

Continued next page.

The meeting will be conducted in English. However, adequate time is allowed for English/Japanese translations and vice versa.

Each paper will be presented by the author or his designated representative. Questions, and discussions between authors and participants, are encouraged throughout the sessions.

Authors were asked to follow a few simple instructions in preparing their manuscripts, and the result presents a pleasing uniformity in appearance. I thank the authors for the time, effort, and work which they put into the preparation of their papers, and trust they will in turn benefit from their participation in this Mini-Workshop.

I thank Mr. Takashi Iijima, PWRI, Japan Ministry of Construction for organizing the whole of the Japanese response to the Mini-Workshop. I thank Mr. Don F. Martin, Director of Paving, B.C. Ministry of Transportation and Highways, for overseeing the local arrangements for the Mini-Workshop and the Field Trip. I thank the Ontario Ministry of Transportation and Communications for their support, particularly Lorraine Sosney and members of the Editorial Staff of the Policy Planning and Research Branch. And lastly, I thank Mr. Peter Eggleton and members of his staff at Transport Canada, Transportation Research Centre, for their assistance and for their financial support.

William A. Phang, Chairman

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Session 1

Low Temperature Cracking of Asphalt Pavements

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ABSTRACT

Much transverse cracking has recently been observed in asphalt pavements, even in newly-laid ones, in the severe climatic zone in Hokkaido, Japan. In spite of the mild climatic condition in Hokkaido, the general feature of this type of pavement failure is quite similar to that of transverse low temperature cracking in Canada.

Depending upon the location, the service history and the design of the pavement, transverse cracking frequencies in the range of 10 to 200 cracks per kilometre were most common, and the length of the cracked pavements amounted to 1500 km (12.3% of the total trunk roads) in 1978 in Hokkaido.

The field survey conducted by Hokkaido Development Bureau revealed that the mechanism of this type of cracking is related to the ambient air temperature, the rate of the cooling, the contraction and the stress relaxation property of the paving mixture, the adhesion of the asphaltic layer to the adjacent non-asphaltic layer, the heat capacity of the base, etc.

In the laboratory experimental study started in 1978 at Hokkaido University, the direct thermal fracture test was carried out to investigate the stress-strain relationship during the cooling of the asphalt concrete specimen and the fracture temperature in such a manner that the specimen was virtually constrained to maintain its initial length throughout a range of temperature; then the specimen was subjected to cooling at a programmed rate. In this laboratory work, effects of binder consistency (30 to 400 Pen.), rate of cooling (-3°C to -30°C/h), and initial starting temperature (10°C to -10°C) were investigated. The stress-strain curve was analyzed from a viewpoint of the stress relaxation property of the asphalt concrete, and it was found that the stress-strain curve is composed from two parts of the stress relaxation zone and the non-relaxation zone. This result suggested that the fracture temperature of the asphalt concrete mainly depends on the magnitude of the relaxation zone.

This paper describes findings of the field survey and the laboratory work in Hokkaido, Japan.

LOW TEMPERATURE CRACKING OF ASPHALT PAVEMENTS

by
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1/ INTRODUCTION

A great amount of transverse cracking recently appeared in asphaltic pavements, even in newly-laid ones, in severely-cold regions in Hokkaido, Japan. It is one of the technically-serious problems. For a period after the occurrence of low temperature transverse cracking, the riding quality of the pavement is not afffected. With time, however, many types of pavement failure generally develop which increase surface irregularity and spalling in the vicinity of cracks. The intrusion of water causes loss of load-carrying capacity of the base and subgrade and reduces the service life of the pavement.

The cracking arising from temperature-induced stress is strongly attributed to the stress relaxation properties of asphaltic mixtures. Thus, this type of cracking is often seen in the pavement in cold regions and, also, on the roads in warm areas where sudden changes in the temperature of pavements occur due to such causes as ambient air temperature and rain, especially when they are combined with properties of asphaltic mixtures.

The type of cracking that is commonly experienced consists of regularly-spaced (20 m to 30 m) transverse cracks in association with longitudinal cracks along the center line or construction joint which provides a weak point in the pavement structure. They are frequently observed at sites where the kneading action by traffic is not so severe. Therefore, this type of pavement failure is regarded as quite different from the cracking attributed to frost heave generally observed in the asphalt pavements in cold regions.

The cracking of asphalt pavement due to temperature-induced stress is a problem of major concern to highway engineers and highway authorities in Hokkaido and, also, in countries such as Canada, United States and northern Europe and is, and has been, a subject of concern for many researchers [1, 2, 3 and 4].

The authors have carried out both field surveys and basic investigations as a joint research project of the Hokkaido Development Bureau and Hokkaido University, and have reached some conclusions on the mechanism of failure of pavement arising from low temperature transverse cracking. The authors suggest that this cracking behavior involves complex factors including ambient temperature and its changes in winter, contraction coefficient and stress relaxation properties of asphaltic mixtures, properties of asphalt cement, and adhesion between the asphaltic layer and the underlying granular layer.

2/ FIELD INVESTIGATION ON LOW TEMPERATURE CRACKING

Extensive field surveys on low temperature transverse cracking of asphalt pavements were conducted on the National and Prefectural Roads of Hokkaido; characteristics of low temperature cracking and causative factors are summarized as follows [5, 6 and 7].

2.1/ Survey of Cracking in Asphalt Pavement

2.1.1/ Statistics of Cracked Pavements

Table 1 shows the statistics of thermally-cracked roads in Hokkaido, in 1978. The cracks are classified into three categories: those with intervals of less than 10 m, 10 m to 30 m and 30 m to 100 m. Low temperature cracking was found in 1486 km out of 12 072 km, at the rate of 12.3%, in trunk roads in Hokkaido.

2.1.2/ Pattern of Low Temperature Cracking

As shown in Figure 1, this type of cracking occurs across the least cross-sectional area of the pavement and appears at regularly-spaced intervals of 20 m to 30 m. In the subsequent period, additional cracks are formed between the original cracks. Typical low temperature cracking appears from the small cracks of the traffic paint in the centre or side lines, the marks of core sampling for density control of asphaltic mixtures, and the corners of gullies near the edge. They are considered to be weak points in pavement structure. The cracks occurring in the surface of the pavement gradually penetrate into lower layers. These observations suggest that the temperature-induced stress does not distribute uniformly over the pavements but concentrates on the defects of the surface, which become the trigger of cracking.

These cracks do not disappear even at high temperatures in summer. Opening of cracks may be narrower under heavy traffic; they generally become wider with time. The survey on the aged pavement which served traffic for more than 10 years has revealed examples where some cracks generate even in summer. Therefore, the initiation of cracking is not restricted to winter. Figure 2 shows a typical cracking pattern, and Photos 1, 2, 3, 4 and 5 show the various types of low temperature cracking in Hokkaido.

2.1.3/ Air Temperature

Figure 3 illustrates the relationship between the distribution of low temperature cracking and freezing indices. The cracks are classified into three categories: those with intervals of less than 10 m, 10 m to 30 m, and 30 m to 100 m, but only those that extend over the whole width of pavements for more than 1 km on the National Roads of Hokkaido area are included in these statistics. This figure clearly indicates that the cracks start most frequently in those areas where the maximum freezing index exceeded 1000 degree-days Celcius for the past 10 years. However, it is emphasized that the freezing index is not the main factor of the initiation of cracks. Pavements with the same pavement structure and freezing indices do not necessarily show the same crack frequency. Table 2 summarizes the relationship between the freezing index, the minimum temperature, the temperature gradient, and the number of cracks on roads constructed in 1977. Based on this table, the initiation of cracking by temperature-induced stress seems to be related more closely to the lowest air temperature and the rate of cooling in the severe winter season rather than the freezing index.

2.1.4/ Age of Pavements and Traffic Conditions

Figure 4 shows the relationship between the number of locations with low temperature cracking observed over 1 km lengths on National roads in the Hokkaido area and the age of pavements. This figure clearly shows that a definite relationship exists between pavement age and cracking. The road survey revealed a close dependence of the crack intervals on the traffic volume. Namely, heavy traffic leads to narrow intervals of cracks. The increase in the frequency of cracks with the age of pavement must be the result of a complex interrelationship between thermal fatigue of pavements caused from repeated changes in air temperature, hardening of asphalt cement and fatigue by traffic loads.

The asphalt-mortar surfacing on National Road No. 38 at Minami-Furano (two-lane, 5.5 m width) showed less than half the number of cracks at the centre line compared to those of the side lines. Generally speaking, vehicles tend to run closer to the centre line. Consequently, the low temperature cracks are closely connected with kneading actions by traffic and the extent of adherence of the asphaltic layer to the underlying granular base.

2.1.5/ Materials of Pavement Components

The relationship between the properties of paving materials and low temperature cracking is analyzed below.

1/ Type of Paving Mixtures

Asphalt mortar, fine-graded asphalt concrete (Topeka), dense-graded gapped asphalt concrete, and fine-graded gapped asphalt concrete have long been used as the surfacing layer of National Roads in the Hokkaido area. Figure 5 illustrates intervals and crack frequency for each of the surfacings mentioned above. The number of cracks with intervals of less than 10 m is proportional to the amount of asphalt cement and filler in surfacings and increases in the following order:

dense-graded gapped asphalt concrete, fine-graded gapped asphalt concrete, fine-graded asphalt concrete, and asphalt mortar.

A field survey was carried out on the National Roads of Hokkaido to compare the pavement types generating a large number of cracks at intervals of less than 10 m with those giving only a small number or no cracks. No significant relationship could be seen between the amount of asphalt cement and the filler passing 74 μ sieve contained in surfacing, binder course, and asphalt-treated base, and the number of cracks. However, it was noted that the number of cracks increases with water absorption of coarse aggregates larger than 5 mm.

2/ Characteristics of Subgrade and Base Materials

In the four sites where the survey revealed frequent low temperature cracking, pavement components were removed to study the effect of subgrade and base materials on cracking. The results are as follows.

- Subgrade soils consist of silty sand and gravel which are known to be non-frost susceptible.
- Pit-run gravel and volcanic ash which compose base and anti-frost layers, respectively, were non-frost susceptible. Both of these materials have no relationship to low temperature cracking.
- In the cracked area, most of the base materials were pit-run gravel smaller than 50 mm, as shown in Figure 6. Density of the base in these locations showed an unexpected increase. These results suggest that adherence of the asphaltic layer to the base is inferior in gravel base.

It is clear that low temperature cracking tends to appear when the materials of the subgrade and base are non-frost susceptible. Once the cracking occurs, density of the base decreases, causing a decrease in the load-carrying capacity of the base and separation of the asphaltic layer from the base.

2.1.6/ Crack Opening

Field observations were conducted to clarify the seasonal variations of pavement contractions, and crack openings were measured throughout the year in severely-cracked areas. The results of this study are shown in Figure 7. Close relationships occur between crack opening and pavement temperature at temperatures higher than 20°C. Such behavior can be explained from the stress relaxation properties at higher temperatures. In other words, the restraining of the surfacing layer to the underlying layer overcomes the elongation of the surfacing layer at high temperatures, while there is a linear contraction of the asphaltic layer in which the material stiffness increases depending on the lowering of temperature in a low temperature range.

Thus, crack opening gradually increases with each consecutive year by the repetition of contraction in winter and restraining in summer. A value of 5 m difference in crack opening throughout a given year was measured, and a value of 15 mm maximum opening was reported in an extremely low temperature.

2.1.7/ Pavement Deflection

It can be estimated that the increases of pavement deflection which cause the reduction of pavement service life increase with the progression of cracking. Spalling was often observed near the cracks of wheel paths in heavy-traffic pavements. To study the loss of the load-carrying capacity of pavement, deflection studies were conducted. Figure 8 shows the results obtained from this study; larger deflections on both inner and outer wheel paths are seen in severely-cracked pavement. Increases in Benkelman Beam deflection were shown in the rates of 25% and 40% and, also, higher deflection values were shown in older pavement.

2.2/ Mechanism of Crack Initiation

The authors propose a mechanism of crack initiation based on the field observation and laboratory investigations on thermal and mechanical properties of asphaltic mixtures. The details are discussed below.

2.2.1/ The Crack Initiation

When asphalt pavement in a large area of roads is in complete restraint to the base and suffers sudden cooling by air temperature, thermal stress develops in the asphaltic layers. Under normal conditions, temperature-induced stress will disappear with time according to the stress relaxation properties of the asphaltic mixtures and will not bring about any change in length. However, when the pavement is cooled rapidly in low temperatures, an extremely large stress develops. Such a large stress will not readily disappear and, as a result, the asphalt will tend to contract. On the other hand, the friction between the asphaltic layer and base will resist the contraction. As a consequence, a tensile force acts on the mixtures which are then under tensile stress. If the stress or strain thus developed exceeds the limiting value of the mixtures, low temperature cracking appears. This mechanism is essentially the same as that proposed for cement concrete pavements. The only difference is that the temperature-induced stress of an asphaltic mixture during cooling would be relaxed before fracture could occur. However, even if the sudden cooling does not lead to crack initiation, repetition of thermal stress in pavements eventually generates fatigue in asphaltic mixtures.

2.2.2/ Factors Influencing the Crack Initiation

In the mechanism of crack initiation presented above, the following factors are related to low temperature cracking:

- contraction coefficient of asphaltic mixture,
- friction coefficient between the asphaltic layer and the base,
- stiffness modulus and stress relaxation properties of asphaltic mixtures,
- temperature gradient and temperature of asphaltic mixtures, and
- tensile strength and permissible strain of asphaltic mixtures.

The adhesion between the asphaltic layer and the granular base may be eventually judged by the frictional coefficient. It ranges from an almost perfect friction wherein the contraction of the mixture leads to contraction of the base, to a completely loose state wherein the asphalt layer can contract independently from the base.

The stress which develops during changes in temperature is strongly influenced by the stiffness modulus of mixtures. The stiffness modulus is a function of loading time and temperature and in the order of 10^5 kg/cm^2 at extremely low temperatures. The asphaltic mixture is visco-elastic material and is capable of relaxing the stress. This relaxation is larger at higher temperatures but decreases with falling temperature. The stress relaxation property of the mixture can be expressed by the length of time in which the applied stress decreases to a certain level. Generally speaking, the stress becomes more and more readily relaxed as the amount of asphalt content increases. However, the coefficient of contraction also increases with its contents: a complex interrelationship exists. Furthermore, the investigation revealed that low temperature cracking tends to increase with the drop in temperature (rate of cooling) and with the increase in viscosity of asphalt cement used.

3/ EXPERIMENTAL WORKS ON TEMPERATURE-INDUCED STRESS AND FRACTURE

3.1/ Introduction

A laboratory investigation has been conducted to evaluate the asphalt properties in temperature-induced stress and fracture in conjuction with the field survey in Hokkaido.

Various reports have been given in this field over a long time in Canada and the United States [8, 9, 10, 11 and 12].

Most of the previous work has shown that the frequency of low temperature cracking is apparently associated with asphalt hardness when identified by original penetration and its temperature susceptibility.

In this study, the fracture behavior of asphalt concrete under various loading conditions at low temperatures and the thermal fracture behavior were investigated.

3.2/ Mechanical Properties of Asphalt Concrete at Low Temperatures

As basic research for the thermal fracture test on asphalt concrete, mechanical properties of asphalt concrete were investigated at low temperatures by means of a beam flexure test. In this study, 15 types of asphalt cement having different penetration grades and temperature susceptibilities were utilized. The size of the specimen was 2.5 cm by 2.5 cm by 25 cm, and the three-point bending in a span of 20 cm was employed. Table 3 shows a composition of asphalt concrete utilized in this investigation. The rate of strain of the flexure test is from the maximum rate of 1.88×10^0 to a minimum rate of 1×10^{-4} 1/sec; the test temperatures were widely varied from -20°C to 30°C.

Three primary responses of asphalt concrete were of interest to this investigation. Failure was defined as the point at which the maximum load ocurred. The flexural strength, the strain at break and stiffness modulus versus the rate of strain as shown in Figure 9 were plotted; this relationship is illustrated in the master curves in this figure.

The general realationships between flexure strength and rate of strain, as well as between strain at break and rate of strain are as shown in the curves. The peak point of strength - rate of strain curve was defined as Transition Point, T_f, by the author; in left hand side of T_f, the fracture mode is brittle and in the right hand side is ductile.

Similar curves also can be obtained from strength versus temperature relationships; in this case T_f can be expressed by temperature. T_f may move to lower or higher rate of strain side, or lower or higher temperature side depending on certain variables; these variables were test temperature or rate of strain, penetration grade of asphalt cement and temperature susceptibility of asphalt cement.

The flexure strength at T_f reaches 110 kg/cm^2, while the strength is between 90 - 100 kg/cm^2 in the brittle fracture zone.

The strain at break versus rate of strain curve shows a gentle S-shaped curve which also moves in the same shape completely corresponding to the movement of the T_f. The lowest strains at break in the brittle fracture zone should always converge at 10^{-3} cm/cm; on the other hand, they are between $2 - 4 \times 10^{-2}$ cm/cm in lower rates of strain or higher temperatures. These data apparently show that there is a considerable difference for a factor of 10 of strain in the brittle and ductile fracture.

The fracture stiffness versus rate of strain is shown as an inclined curve; the highest stiffness values also should always converge at 1×10^5 kg/cm^2.

The tests mentioned above were applied for asphalt concretes composed of various types of asphalts, and it was found that the mechanical properties of asphalt concrete at the Transition Point are as follows:

Strength at Transition Point	115 ± 15 kg/cm^2
Strain at Break	$(5 \pm 1) \times 10^{-3}$ cm/cm
Fracture Stiffness	$(2 \pm 1) \times 10^4$ kg/cm^2

Ductile fracture means the presence of stress relaxation during loading, while there is no stress relaxation in the brittle fracture zone. From this viewpoint, T_f might be considered as a boundary of stress relaxation, and the mechanical property at this point is defined as a point at which the stiffness modulus reaches $(2 \pm 1) \times 10^4$ kg/cm^2.

This stiffness value itself is independent of loading time. This concept of Transition Point might be meaningful to the study of temperature-induced stress and fracture.

Figure 10 shows relationships between Transition Point, T_f, versus softening points of asphalt cement; parameters used are penetration of original asphalt cement and Penetration Index. Dotted lines show the experimental data obtained from the beam flexure tests at the rate of strain of 1.88×10^{-2} 1/s. These Transition Points move 6°C according to the change of one order of the rate of strain.

These test results can be used for estimation of the Transition Points. An estimation of the Transition Point becomes possible by using the stiffness modulus of asphalt cement, S_{bit}, and the stiffness of asphaltic mixture, S_{mix}, relationship and the stiffness modulus of asphalt cement which was obtained from the van der Poel nomograph [15].

In this study, a value of 2 x 10^2 kg/cm^2 of asphalt cement which corresponds to a value of 2 x 10^4 kg/cm^2 of asphaltic concrete, based on the author's previous work on the creep behavior of the asphalt concrete, was used [16].

In Figure 10, the solid lines show the equi-stiffness lines of 2 x 10^2 kg/cm^2 at a loading time of 0.27 s, 5 x 10^{-3}/1.88 x 10^{-2} s.

In general, both graphs hold the same position, but there are some differences between experimental values and calculated values from the van der Poel nomograph in a higher Penetration Index range.

These test results clearly show that the Transition Points could be predicted from the van der Poel nomograph with a reasonable degree of accuracy.

3.3/ Thermal Fracture for Asphalt Concrete

3.3.1/ Test Method

A series of experiments was carried out to measure temperature-induced stress and fracture temperature of a beam specimen of asphalt concrete. The composition of the asphalt concrete is given in Table 3. A specimen measuring 2.5 cm by 2.5 cm by 26 cm was virtually constrained to maintain its initial length throughout a range of temperatures. Then the specimen was subjected to cooling at a programmed cooling rate. The loads associated with the temperature lowering were measured by a load cell.

A specimen coated with thin emulsified rubber to avoid water penetration was cemented with epoxy resin to steel end-pieces which were connected with a rigid steel frame through the extension rods made of invar steel. A water bath filled with methanol was placed in such a way that the invar extension rods passed through both end-walls of the water bath. The load cell was placed at the end of the extension rod at the outside of the water bath. Figure 11 and Photos 6 and 7 show the test equipment. This equipment consists of a measuring device with a water bath, cooling unit and control unit. The lowest measurable temperature in this equipment is set at -39°C at a cooling rate of 12°C/h.

This type of test is not devoid of trouble, and scattering of the observed values is unavoidable to a certain extent. The causes of the scattering might be attributed to a 'fluctuation' depending on the fracture mechanism itself at low temperatures, and experimental errors. A considerable fluctuation was seen in this study; such fluctuations are unavoidable in fractures at low temperatures.

3.3.2/ Temperature-Induced Stress Versus Temperature and Strain

Figure 12 is a schematic representation of temperature-induced stress versus temperature relationships obtained from direct thermal fracture tests. There is no shrinkage in the restrained specimen, and the lowering of temperature causes an elongation in the specimen. This elongation can be calculated from the temperature difference and the thermal contraction coefficient of the specimen. A value of 2×10^{-5} 1/°C for the coefficient was chosen based on the thermal contraction test in this study.

The slope of this illustrated stress-strain curve gradually increases as the temperature goes down to a 'certain' temperature; beyond this critical temperature, the slope of this curve reaches a constant value and a straight line is seen. It is certain that the temperature-induced stress is relaxed down to this temperature, while there is no stress relaxation in a straight line zone. In this study, this particular temperature was defined as Transition Point, T_t. The presence of this particular point shows that the stress-strain curve consists of two zones of relaxation and non-relaxation where the strain is independent of time; thus, the Transition Point is critical. It may be expected that this Transition Point, T_t, corresponds to the Transition Point, T_f, obtained from the beam flexure test.

3.3.3/ Thermal Fracture Test of Asphalt Concrete with Various Types of Asphalts

The stress-strain curves of asphalt concrete utilizing various types of asphalts were obtained to investigate the influences of the properties of asphalt. Twenty-three types of asphalts over a wide range of penetrations, softening points and temperature susceptibilities were employed. Table 4 shows the test results. The following were tabulated:

- properties of asphalt;
- temperature-induced stress, temperature and strain at the Transition Point;
- fracture temperature, fracture strain and fracture strength;
- temperature difference between Transition Point and fracture; and
- slope of the non-relaxation zone which means the stiffness modulus.

In this study, relatively large scattering values were observed. It was noted that these scattered values were much larger in temperature at the fracture, and scattering is larger for harder asphalts compared with softer asphalts. This scattering sometimes reached about 3°C. In this study, representative values used for further discussion were, in most cases, average values.

The fracture strengths are somewhat scattered, but most of the fracture strengths are distributed between 25 and 30 kg/cm^2. Small values were seen in the harder asphalt. There are significant differences in fracture temperatures and fracture strains depending upon the asphalt properties.

The stiffness moduli were distributed between 6×10^4 and 8×10^4 kg/cm^2 except for three asphalts having a small penetration. The particular value of 6 to 8×10^4 kg/cm^2 as the stiffness modulus at the Transition Point is appropriate for an exploration of the thermal property of asphalt concrete. If the movement of the Transition Point was clarified, useful information might be provided for this study. This particular stiffness value means that the temperature difference between the Transition Point and fracture point is "10°C" and "2×10^{-4} cm/cm" in strain.

Further analysis is needed to clarify more detailed thermal behavior.

3.3.4/ Effect of Penetration Grades of Asphalts

Figure 13 shows the test results of the effects of penetration grades of asphalts. In this figure, asphalt concrete penetrations 34 to 470 have a Penetration Index from -1.1. to -0.4 except for penetration 382 and 470. Temperature-induced stress versus temperature was plotted for eight types of asphalts and the Transition Point and fracture were also plotted in this figure. Each curve is situated in a regular order of penetrations. The values of the slope in the non-relaxation zone are quite similar except for penetration of 470 asphalt, and are shown as (7 ± 1) $\times 10^4$ kg/cm^2. There were no significant differences in the fracture strength, except for penetration 34 asphalt, while there were considerable differences in the fracture strain. These differences in the fracture strain completely depend on the magnitude of the relaxation zone.

3.3.5/ Effect of Temperature Susceptibility of Asphalt

A few experiments were performed to investigate the effect of the temperature susceptibility of asphalt. The fracture temperature of the asphalt concrete depends on the penetration grade, but there are some differences in the fracture temperature for asphalts having different temperature susceptibility even in the same penetration grade.

Figure 14 compares five asphalts having a Penetration Index from -0.5 to +1.1 in the same penetration grade. The five curves are arranged in order of the Penetration Index. These test results clearly show a lower fracture temperature for a higher Penetration Index. A difference in the slopes in the non-relaxation zone was not found in this study.

From Figures 13 and 14, it is clear that the fracture temperature of the asphalt concrete depends on both its penetration and temperature susceptibility.

3.3.6/ Effect of Cooling Rate

To investigate the effect of the cooling rate on temperature-induced stress and fracture temperatures, a few experiments were conducted. The starting temperature of 10°C and six rates of cooling between 3°C and 30°C /h were employed.

Table 5 shows the test results. From the table it can be considered that there is no significant effect of the cooling rate on the fracture temperature and fracture strain. If they are plotted as stress-strain curves, all six curves are placed in the same position.

Although, generally, the fracture temperature at lower cooling rates, which means a longer loading time, shows a lower fracture temperature depending on the widening of the relaxation zone, in this case such a tendency was not found. This seems to be influenced by other factors which have not been considered. While the lower fracture strength was shown in the lower cooling rates, this lowering of strength may suggest the initiation of micro cracking and its progress in the specimen prior to the fracture. Micro cracking may increase with cooling time.

As a result it is felt that the effect of the cooling rate on the fracture temperature is relatively small.

3.3.7/ Effect of Starting Temperature

To investigate the effect of the starting temperature of cooling, some tests were performed. The starting temperature varied from -10°C to +20°C.

In this investigation, satisfactory results could not be obtained because of the difficulty in setting up the specimen on the test equipment at low starting temperatures. Thus, it was necessary to make a few corrections in the measured values.

Test results are shown in Table 6. The corrected values from the stress-strain curves are also indicated in this table. It will be seen that varying the starting temperatures between -10°C and +20°C had no significant effect on the fracture temperature itself. While there were some differences in the fracture strength, lower strength at lower starting temperature was observed. Fracture strains were lowered with the lowering of the starting temperature depending on the magnitude of the relaxation zone. The lowest strain was reported as 4.5×10^{-4} cm/cm.

This result suggests that the possible lowest strain may attain 2×10^{-4} cm/cm depending on the starting temperature.

3.4/ Prediction of Probable Fracture Temperature

As a result of direct thermal fracture tests, the following conclusions were reached.

- The temperature-induced stress versus strain curve is constituted from two zones of stress relaxation in which a stress relaxation can be expected and a non-relaxation zone where the stress is independent of time.

- The boundary of these two zones was defined as the Transition Point, T_t, at which the stiffness of asphaltic mixture, S_{mix}, reaches $(7 \pm 1) \times 10^4$ kg/cm^2. This Transition Point moves regularly depending on the asphalt properties.

- The magnitude of the non-relaxation zone is 10°C in temperature and 2×10^{-4} cm/cm in strain.

- The fracture strength at low temperatures was distributed betwen 25 and 30 kg/cm^2. This value is 30% of the beam flexure strength and 50% of the previously-reported tensile strength.

  This may suggest the progressive initiation of the micro cracking in the specimen during the long-term cooling.

- Asphalt properties are dominant influences on the fracture temperature which depends on both the penetration grade and the temperature susceptibility.

- Significant effects of starting temperature and cooling rate were not found.

The preceding test results can be used for the estimation of the fracture temperatures of asphalt concrete constituted from various types of asphalts.

This approach is basically a variation of the limiting stiffness approach based on the direct thermal fracture test and the van der Poel stiffness nomograph [15]. Only two steps are required, the first step being the determination of the Transition Point. The Transition Point is defined as the temperature at which the mixture stiffness, S_{mix}, shows a value of 7×10^4 kg/cm^2. This S_{mix} is independent of the loading time. When an appropriate loading time is provided, an estimation of the Transition Point becomes possible from the uses of the S_{mix}-S_{bit} relationship and the stiffness of asphalt, S_{bit} , which was obtained from the van der Poel nomograph.

In this study, a value of 7×10^2 kg/cm^2 of S_{bit} was adopted corresponding to a value of 7×10^4 kg/cm^2 of the S_{mix}, based on the author's previous work [16] concerned with the creep behavior of the asphalt concrete. These S_{mix}-S_{bit} relationships of various asphaltic mixtures can also be calculated easily from Heukelom's work [17].

In the case of thermally-induced stress or thermal fracture, there is some difficulty in the setting of an appropriate loading time because the temperatures fall with time during the test. For this reason, the cooling time cannot be used for the loading time. From the above consideration, the loading time was calculated back from the van der Poel nomograph, through the Transition Point, T_t, and the value of 7×10^2 kg/cm^2 of S_{bit}. Thus, a value of 100 s as the loading time was obtained. And then, as a second step, the fracture temperature was calculated as the T_t minus 10°C.

The results of such estimations for a wide range of asphalts are presented in Table 7. The estimated and measured temperatures at both Transition

Points and fracture points were tabulated. Figure 15 shows the relationships between the measured and estimated temperatures. This estimation is in fairly good agreement with the experimental values. In the cases of penetrations 382 and 470, reasonable results were not obtained; this may depend on the capacity of the test equipment, where the limitation was given as -39°C in this estimation method.

The prediction nomograph was made in the same manner as in Figure 10 and is shown in Figure 16. In this nomograph, both the predicted Transition Point and fracture temperature were scaled on the vertical axis and the softening point of asphalt cement was scaled on the horizontal axis, and two parameters of penetration and Penetration Index were employed. The measured values were not plotted to avoid confusion; actually both lines were situated in the same position. This nomograph is valid for asphalt concrete having 0.84 to 0.86 for the volume concentration of aggregate.

4/ CONCLUSIONS

An extensive field survey and basic research on the low temperature cracking of asphalt pavements were conducted. The principal points may be summarized as follows.

1/ Low temperature cracking was found in 1486 km out of 12 072 km, at a rate of 12.3%, in trunk roads in Hokkaido in 1978.

2/ Low temperature cracking increases with the age of the pavement, but was found even in newly-laid ones.

3/ Low temperature cracking is considerable in pavements which were laid on pit-run gravel base.

4/ Significant decreases of bearing value were reported in the cracked area and the loss of bearing value might have brought the rapid reduction of pavement life.

5/ Laboratory investigation has shown that the fracture trmperature of asphalt concrete depends both on its penetration and its temperature susceptibility.

6/ The prediction nomograph was constructed based on the direct thermal fracture tests and the van der Poel stiffness nomograph. This nomograph is in fairly good agreement with thermal fracture test results.

7/ 80/100 penetration asphalt has been used in Hokkaido. This study suggests that softer asphalt is preferable to eliminate low temperature cracking.

The cracking of asphalt pavement due to thermal effects is a problem of major concern to highway engineers in colder climates. Considerable further investigation of both field and laboratory aspects must be done. In Japan, the research on low temperature cracking has just started and there still remains a wide unexplored domain.

ACKNOWLEDGEMENTS

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Table-1   Low Temperature Cracking in Hokkaido.
(based on 1978 Survey)

Class of Roads	Paved Roads (km)	Crack Interval (m)			Total	
		<10 (km)	10 - 30 (km)	30 - 100 (km)	Length (km)	%
National	4 974	250.1	184.7	407.5	842.3	16.9
Prefectural	7 098	278.9	163.7	197.6	643.8	9.1
Total	12 072	529.0	352.0	605.1	1 486.1	12.3

Table-2   Relationship Between Change of Air Temperature and Cracking in Winter

Locations*	Freezing Index °C days	Lowest Air Temp. °C	Max. Range of Temp. in a day	Air Temp.Gradient °C/h in Feb.1978		No.of Cracks**
				Average Value	Max. Value	
Horokanai (No.275 Roads)	1150	-38.5	30.5	1.09	2.68	58
Ashoro (No.242 Roads)	1270	-32.5	26.8	1.18	1.91	4
Nukabira No. 273 Roads)	1280	-30.5	20.0	0.91	1.65	0

* : Paved in Oct. 1977   ** : Observed in June 1978

Table-3   Compositions of Asphalt Concrete

Sieve Opening (mm)	Fracture Test Pass, %	Thermal Test Pass, %
13	100	100
10	92	89
7	75	78
5	68	67
2.5	47	48
1.2	36	35
0.6	25	33
0.3	14	23
0.15	7.2	11
0.074	6.0	8.5
Asphalt Content	6 %	5.8 %

Table-4 Results of Direct Thermal Fracture Tests

Rate of Cooling : $12^{o}C/h$
Starting Temperature : $10^{o}C$

No.	Pen.	T_{RdB}	PI	Transition Point			Fracture			Temp*	Slope
				Temp. ^{o}C	Stress kg/cm^2	Strain $x10^{-4}$	Temp. ^{o}C	Stress kg/cm^2	Strain $x10^{-4}$	dif. ^{o}C	kg/cm^2
1	34	54.0	-1.1	- 9.0	8.5	3.8	-19.0	21.5	5.8	10.0	6.0
2	40	58.1	+0.1	-15.7	13.5	5.1	-26.0	28.6	7.2	10.3	7.2
3	44	60.5	+0.8	-17.5	17.0	5.5	-29.0	32.8	7.8	11.5	7.9
4	50	56.5	+0.3	-19.0	15.5	5.8	-26.6	21.3	7.3	7.6	4.7
5	60	50.6	-0.6	-15.0	10.1	5.0	-25.6	20.9	7.1	10.6	5.7
6	67	50.0	-0.5	-17.5	14.7	5.5	-28.3	30.0	7.7	10.8	6.4
7	67	56.5	+1.1	-23.9	16.4	6.8	-34.3	29.7	8.9	10.4	7.2
8	68	53.5	+0.5	-20.5	13.1	6.1	-32.3	28.9	8.5	11.8	7.0
9	69	48.2	-0.9	-16.0	14.0	5.2	-25.4	29.4	7.1	9.4	8.2
10	69	50.5	-0.2	-20.0	15.7	6.0	-28.5	26.0	7.7	8.5	6.6
11	71	51.0	-0.1	-18.5	16.6	5.7	-27.7	26.3	7.5	9.2	6.5
12	71	57.0	-1.3	-25.0	13.6	7.0	-36.4	26.4	9.3	11.4	5.8
13	72	50.5	-0.1	-20.0	17.0	6.0	-28.3	28.0	7.7	8.3	7.1
14	79	46.6	-1.0	-16.3	15.0	5.3	-26.6	29.0	7.3	10.3	7.3
15	86	47.8	-0.4	-18.7	18.7	5.7	-31.0	32.5	8.2	12.3	7.4
16	86	51.0	+0.5	-25.5	21.2	7.1	-33.4	31.3	8.7	7.9	7.5
17	108	44.5	-0.7	-20.0	16.6	6.0	-30.1	29.3	8.0	10.1	7.0
18	131	43.0	-0.6	-22.0	18.7	6.4	-31.1	32.9	8.2	9.1	7.9
19	134	42.8	-0.6	-22.5	17.0	6.5	-33.1	30.5	8.6	10.6	7.2
20	173	40.5	-0.4	-26.0	14.5	7.2	-36.4	30.0	9.3	10.4	7.9
21	235	37.5	-0.4	-29.3	15.0	7.9	-38.4	27.3	9.7	9.1	7.6
22	382	35.5	+1.9	-30.3	15.0	8.1	-39.5	29.5	9.9	9.0	8.0
23	470	34.5	+3.5	-30.5	10.7	8.1	-	-	-	-	6.0

* : Temperature Difference between Transition Point and Fracture Temperature

Table-5 Effect of Rate of Cooling on Fracture

Asphalt : Pen. 86, S. P. 47.8°C
Starting Temperature : 10 °C

Rate of Cooling °C/h	Fracture Temperature °C	Fracture Strain x10^{-4}	Fracture Strength kg/cm^2
- 3	-27.1	7.4	27.3
- 6	-30.2	8.0	27.6
-12	-31.0	8.2	31.0
-18	-31.0	8.2	32.7
-24	-32.0	8.4	33.0
-30	-32.5	8.5	31.8

Table-6 Effect of Starting Temperature on Fracture

Asphalt : Pen. 86, S.P. 47.8°C
Rate of Cooling : 12 °C/h

Starting Temperature °C	Fracture Temperature °C	Fracture Strain x 10^{-4}	Fracture Strength kg/cm^2
-10	-32.3	4.8	23.9
- 5	-29.1 (-31.0)*	4.8 (5.3)*	21.2 (25.0)
0	-23.5 (-33.0)*	4.7 (6.3)*	17.2 (26.5)
5	-33.1	7.6	27.4
10	-30.2	8.2	31.0
20	-33.6	10.7	26.8

*** : Estimated Values Obtained from Stress-Strain Relationship Curves**

Table-7 Relationship Between Estimated and Measured Values at Transition Points and Fractures

No.	Pen.	$T_{R\,d\,B}$	PI	Transition Point Estimated (°C)	Transition Point Measured (°C)	Fracture Temp. Estimated (°C)	Fracture Temp. Measured (°C)
1	34	54.0	-1.1	- 6.8	- 9.0	-16.8	-19.0
2	40	58.1	+0.1	-14.3	-15.7	-24.3	-26.0
3	44	60.5	+0.8	-18.2	-17.5	-28.2	-29.0
4	50	56.5	+0.3	-17.8	-19.0	-27.8	-26.6
5	60	50.6	-0.6	-15.4	-15.0	-25.4	-25.6
6	67	50.0	-0.5	-17.0	-17.5	-27.0	-28.3
7	67	56.5	+1.1	-24.9	-23.9	-34.9	-34.3
8	68	53.5	+0.5	-22.5	-20.5	-32.5	-32.3
9	69	48.2	-0.9	-14.8	-16.0	-24.8	-25.4
10	69	50.5	-0.2	-19.5	-20.0	-29.5	-28.5
11	71	51.0	-0.1	-19.8	-18.5	-29.8	-27.7
12	71	57.0	+1.3	-26.3	-25.0	-36.3	-36.4
13	72	50.5	-0.1	-20.3	-20.0	-30.3	-28.3
14	79	46.6	-1.0	-15.4	-16.3	-25.4	-26.6
15	86	47.8	-0.4	-20.2	-18.7	-30.2	-31.0
16	86	51.0	+0.5	-25.0	-25.5	-35.0	-33.4
17	108	44.5	-0.7	-20.5	-20.0	-30.5	-30.1
18	131	43.0	-0.6	-23.0	-22.0	-33.0	-31.1
19	134	42.8	-0.6	-23.2	-22.5	-33.2	-33.1
20	173	40.5	-0.4	-27.5	-26.0	-37.5	-36.4
21	235	37.5	-0.4	-30.5	-29.3	-40.5	-38.4
22	382	35.5	+1.9	-	30.3	-	-40.5
23	470	34.5	+3.5	-	30.5	-	-

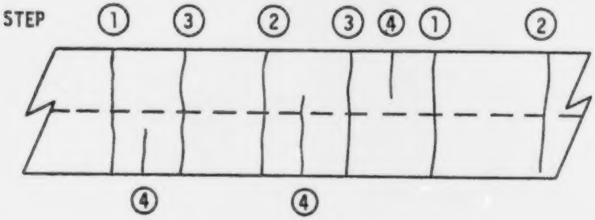
STEP
①
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Figure-1 Pattern of Crack Initiation of Low Temperature Cracking

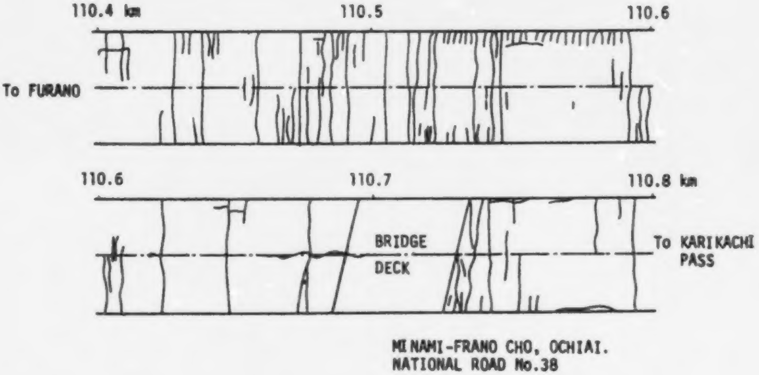

110.4 km
110.5
110.6
To FURANO
110.6
110.7
110.8 km
BRIDGE
DECK
To KARIKACHI
PASS
MINAMI-FRANO CHO, OCHIAI.
NATIONAL ROAD No.38

Figure-2 Typical Pattern of Low Temperature Cracking in National Road No. 38, Minami-Furano-Cho, Hokkaido

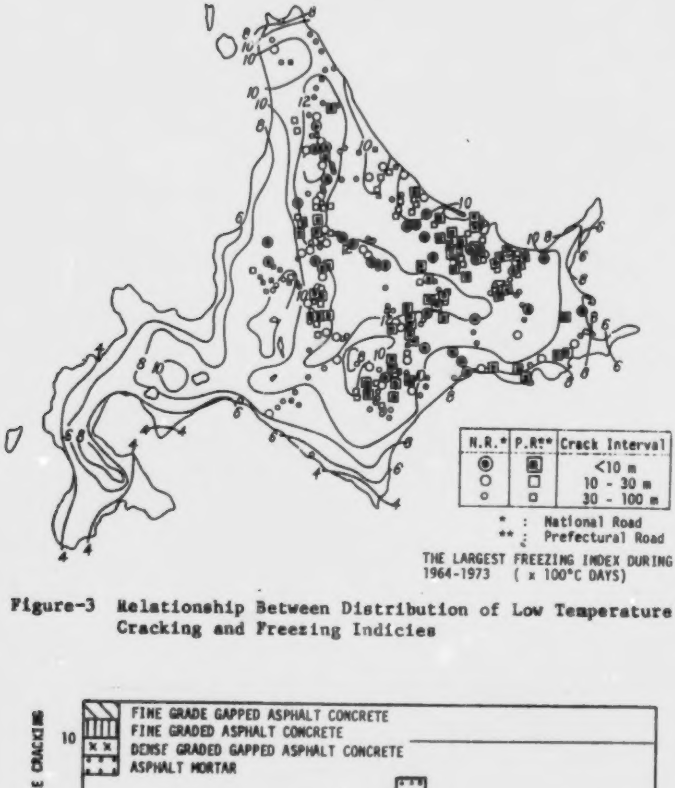

N.R.*
P.R.**
Crack Interval
<10 m
10 - 30 m
30 - 100 m
* : National Road
** : Prefectural Road
THE LARGEST FREEZING INDEX DURING 1964-1973 (x 100°C DAYS)
4
6
8
10
12

Figure-3 Relationship Between Distribution of Low Temperature Cracking and Freezing Indicies

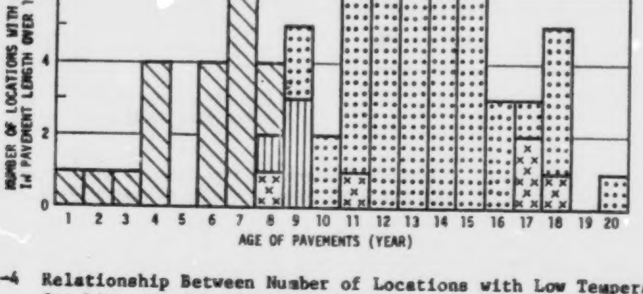


Figure-4 Relationship Between Number of Locations with Low Temperature Cracking and Age of Pavements

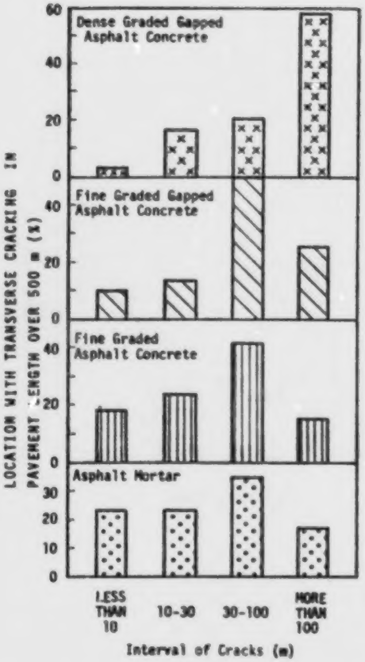


Figure-5 Crack Intervals and Crack Frequency of Various Types of Pavements

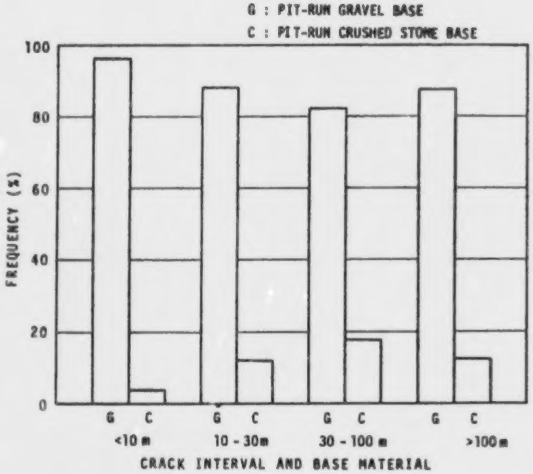


Figure-6 Influences of Base Materials on Crack Frequency

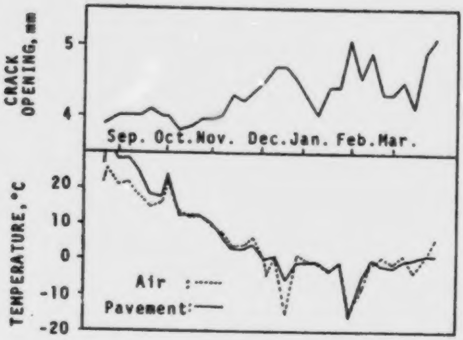


Figure-7 Relationship Between Temperature and Crack Opening

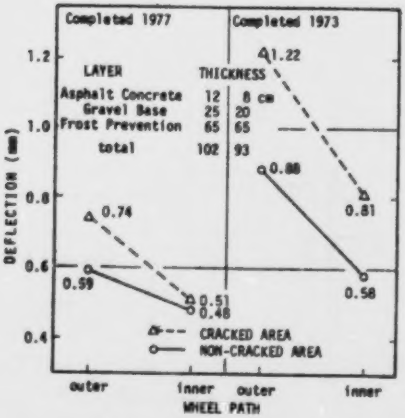


Figure-8 Benkelman Beam Deflection in Cracked and Non-Cracked Pavements

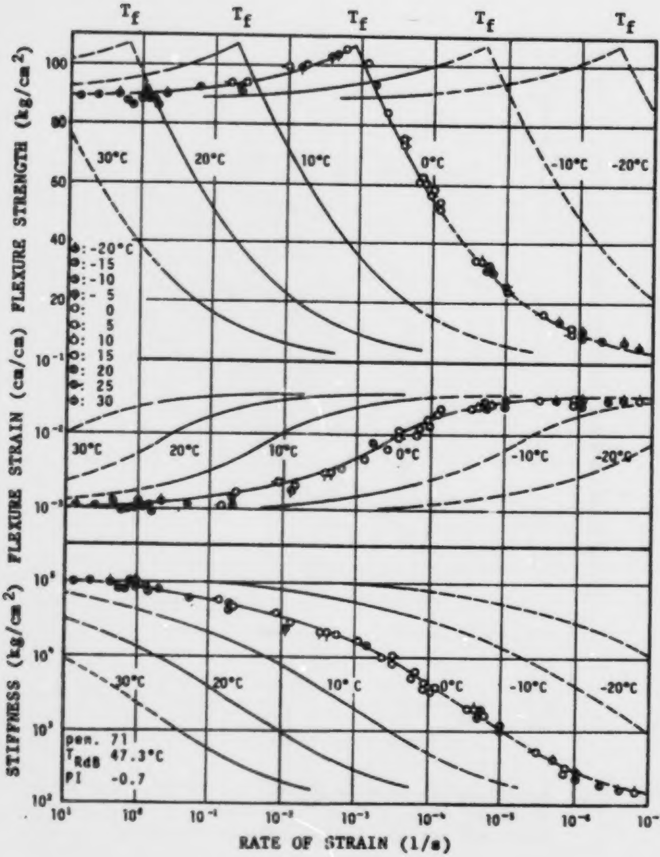


Figure-9 Effects of Rate of Strain on Fracture Strength, Fracture Strain and Stiffness of Asphalt Concrete Obtained from Beam Flexure Tests

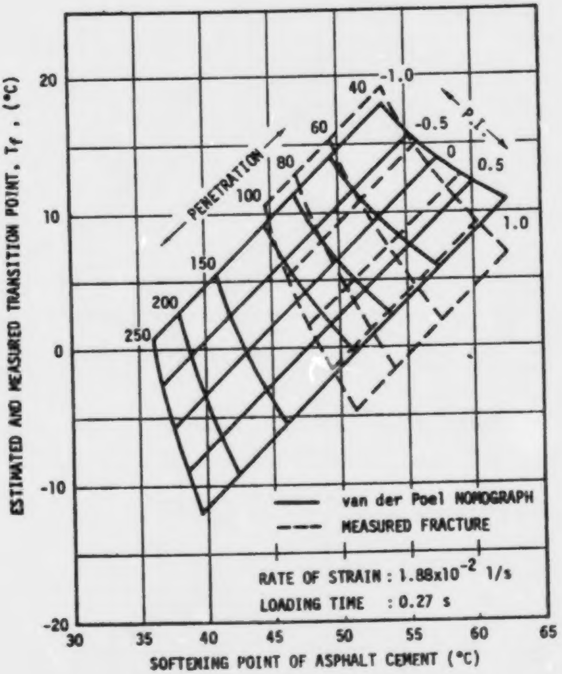


Figure-10 Relationship Between Transition Point, T_f, and Softening Point of Asphalt Cement Obtained from Beam Flexure Test and van der Poel Stiffness Nomograph

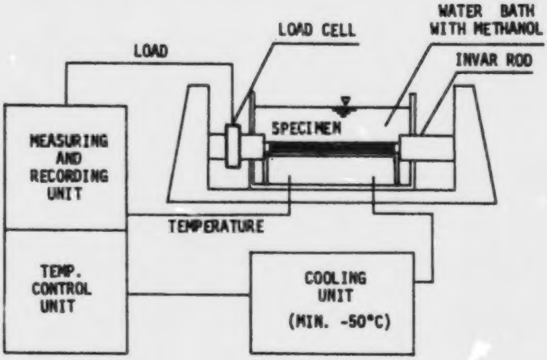

LOAD
LOAD CELL
WATER BATH
WITH METHANOL
INVAR ROD
SPECIMEN
MEASURING
AND
RECORDING
UNIT
TEMPERATURE
TEMP.
CONTROL
UNIT
COOLING
UNIT
(MIN. -50°C)

Figure-11 Schematic Diagram of Test Equipment

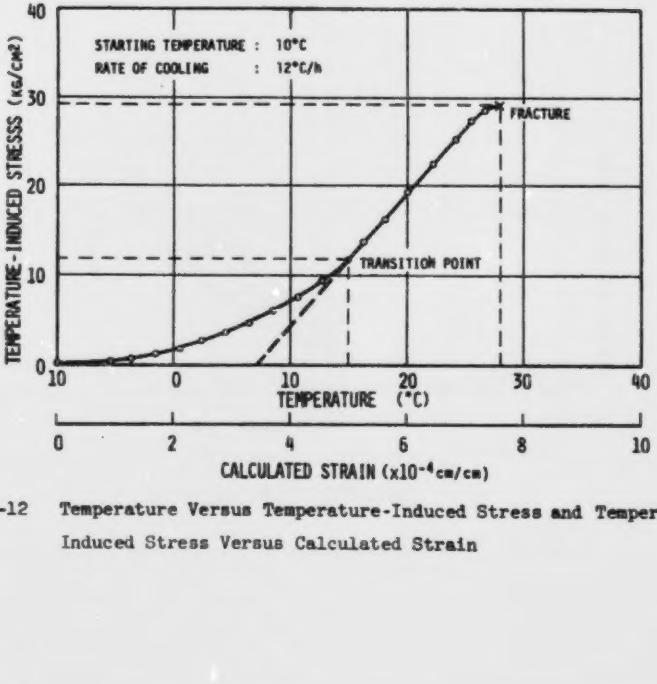
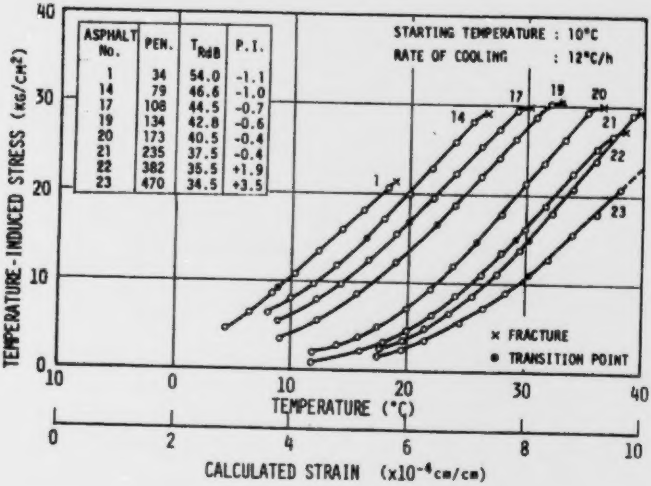
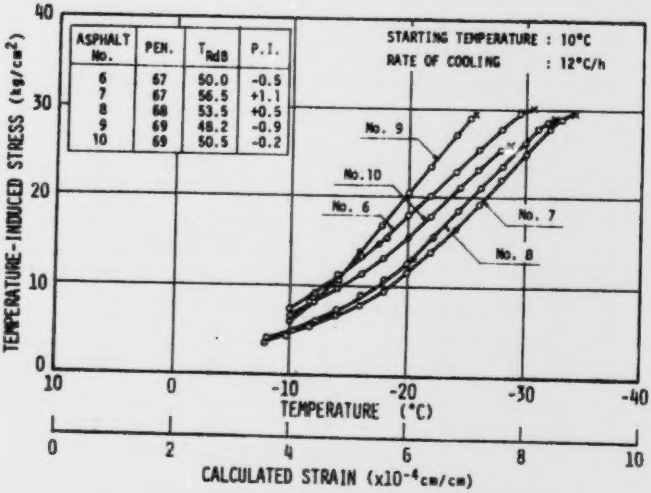


Figure-12 Temperature Versus Temperature-Induced Stress and Temperature-Induced Stress Versus Calculated Strain



ASPHALT No.	PEN.	T_{RdB}	P.I.
1	34	54.0	-1.1
14	79	46.6	-1.0
17	108	44.5	-0.7
19	134	42.8	-0.6
20	173	40.5	-0.4
21	235	37.5	-0.4
22	382	35.5	+1.9
23	470	34.5	+3.5

Figure-13 Effects of Penetration of Asphalt Cement on Temperature-Induced Stress and Fracture



ASPHALT No.	PEN.	T_{RdB}	P.I.
6	67	50.0	-0.5
7	67	56.5	+1.1
8	68	53.5	+0.5
9	69	48.2	-0.9
10	69	50.5	-0.2

Figure-14 Effects of Penetration Index of Asphalt Cement on Temperature-Induced Stress and Fracture

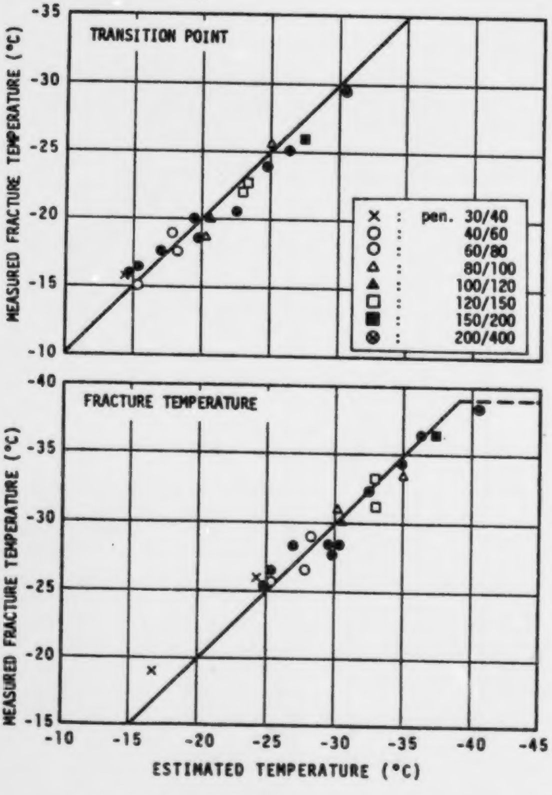


Figure-15 Relationship Between Measured and Estimated Transition Point, T_t, and Measured and Estimated Thermal Fracture Temperature

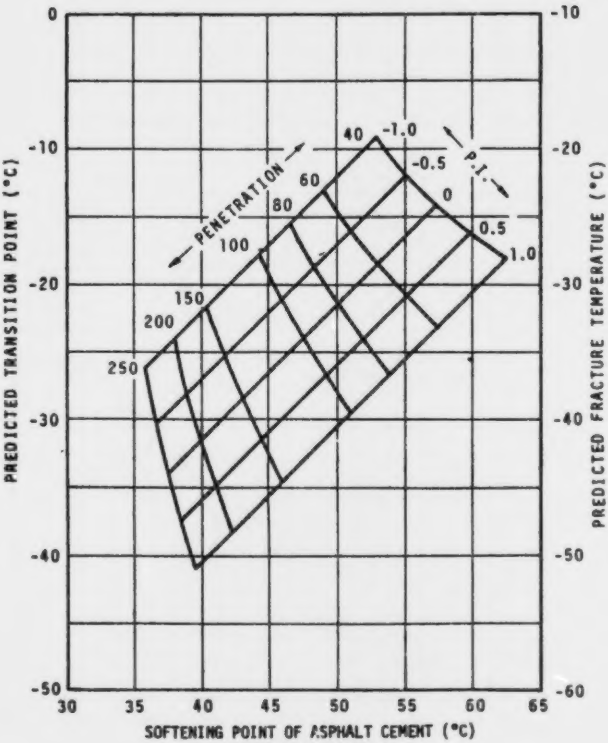


Figure-16 Nomograph for Prediction of Transition Point, T_t, and Fracture Temperature of Asphalt Concrete Obtained From Direct Thermal Fracture Test and van Der Poel Stiffness Nomograph

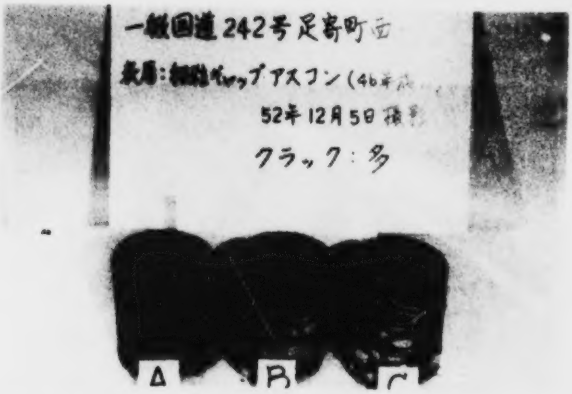

一級国道242号足寄町西
表層:細粒ギャップアスコン（46年施
52年12月5日採取
クラック：多
A
B
C

Photo-1 Core Samples of Cracked Pavement

A : Crack in Full Depth
B : Crack in Surface
C : No Crack

Photo-2 **Typical Low Temperature Cracking in Minami-Furano-cho, Hokkaido**

Photo-3 Progressive Low Temperature Cracking

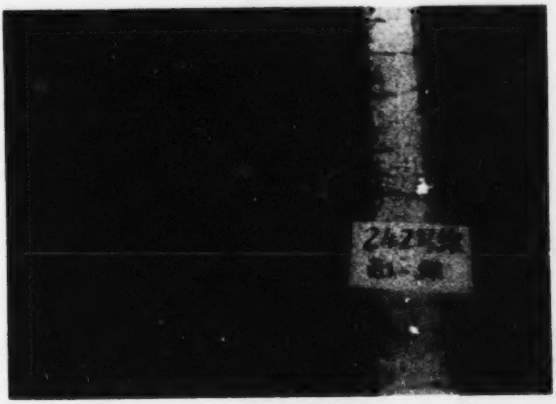


Photo-4 Crack Generated in Shrinkage Cracking of Traffic Paint

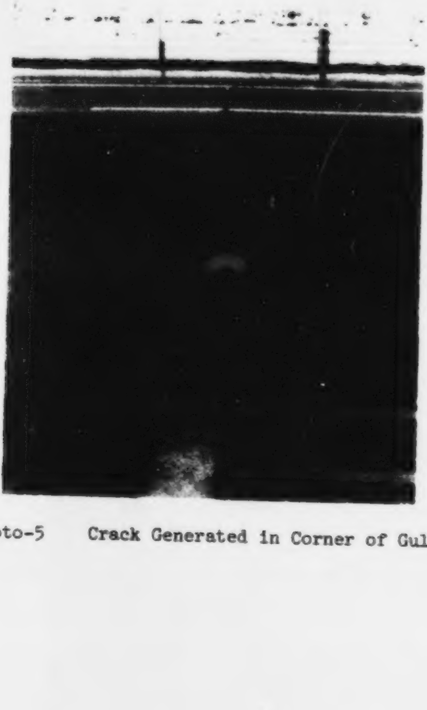


Photo-5 Crack Generated in Corner of Gully

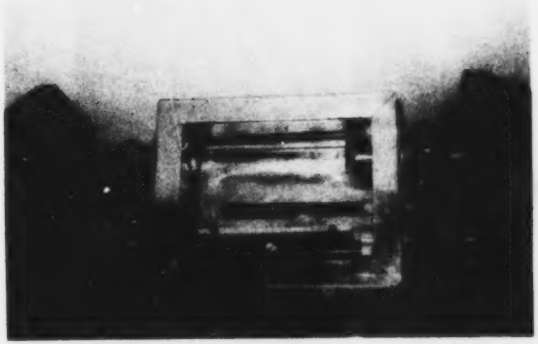


Photo-6 Measuring Device with Water Bath for Thermal Fracture Test

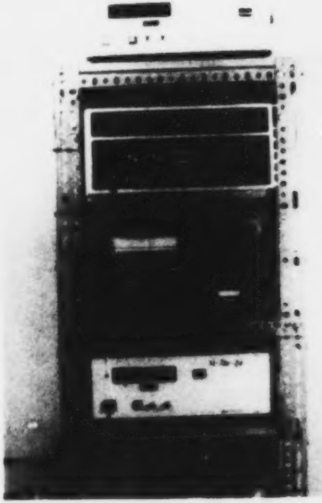


Photo-7 Control, Measuring and Recording Unit for Thermal Fracture Test

Session 1

Low Temperature Cracking of Asphalt Pavements - a Reconsideration

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LOW TEMPERATURE CRACKING OF ASPHALT PAVEMENTS - A RECONSIDERATION

by

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and

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Introduction

Since Rader published his paper on the physical properties of asphaltic materials at low temperature in 1935 (1), several models have been developed to explain low temperature cracking of asphalt pavements (2-7). Although asphalt pavements are multilayered systems, most models just take into account the properties of the asphalt layer, as the cracking mechanism of this isolated layer and the influence of the factors involved are generally well defined in the literature.

A number of factors interrelating the other layers are known to influence low temperature cracking; as cited in reference 7: Carpenter, Lytton and Epps (8) indicated that stresses in the non asphalt treated base layer can cause transverse cracks which ultimatly reflect through the surface; Deme (9) and also Hamilton (10) indicated that transverse shrinkage cracks occur in the subgrade and can propagate through the pavement structure by differential movement and be reflected in the pavement surface. All soil engineers practicing in cold regions know that non-uniformities in the frost susceptible subgrade can cause differential frost heaving of the pavement resulting in cracking.

Recent studies presented experimental evidence based on the evaluation of two northern roads in Quebec (11) and in Alaska (12). They showed that, if the existing models are reliable for predicting low temperature cracking of particular sections, they could not be of general application. Keyser and all (11, 13) for example, are following the performance of 2480 sections of pavement representing 620 Km of road running South-North to the James Bay area; this, through continuous evaluacion of roughness, strength and surface degradation. The pavement is composed of

10 cm of bituminous overlay made with 300-400 pen asphalt, 15 cm base course and 60 cm subbase. The severity and extent of transverse cracking were counted every 2 years and presented by the frequency curve of figure 1. All the characteristics of the bituminous mixture which are used for crack prediction have been defined during or after the construction. Four models have been evaluated with results obtained 4 years after the construction, i.e. USCE or Shahin's model, McLeod's model, Haas's model and the Anderson-Christison's model. The need for a more generalized approach was evident through evaluation and the following conclusions were (11) made.

a) Generally speaking, the 4 models tested worked for 2/3 of the 2480 sections evaluated.
b) For the remaining third, for which the degree of cracking varied widely, the data were analysed, by correspondence analysis which clearly indicated that for the cracked sections of the road, there is practically no relation between the number of transverse cracks and the characteristics of the mix used as design parameter in the models. This is shown in figure 2.
c) In some sections frost cracks which run through the shoulders account for 1/10 to 1/20 of all transverse cracking.

In this paper, the authors will try :

- To identify the elements of a more general approach to the design of pavement to minimize low-temperature asphalt pavement transverse cracking.
- To give the framework of a more general model which can be used for the development of a design method.

This is done for a typical multilayered system of a flexible pavement as defined hereafter.

1 - DESCRIPTION OF THE GLOBAL APPROACH

The concept of a global approach for the design of pavement for minimizing low temperature cracking is illustrated in figure 3. The approach is being based on the following broad considerations.

1 - Pavement is a layered system. Low temperature cracking of pavement can be caused by the performance of a given layer or the combined

effect of several layers.

2 - Local climate is one of the most important factors affecting low temperature cracking. Local climate can only be adequately defined with a full knowledge of the geography of the site, i.e. the geomorphology and the weather conditions.

3 - The characteristics of local road materials and in-situ soils which compose the layered system depend on the local geology and geomorphology.

4 - Transverse cracking of a pavement is mainly caused by non load associated stresses: it can however be the result of the combination with other types of stresses such as stresses induced by settlement and heaving and stresses due to loads.

5 - Local climate, road material and construction, and soils being variable in nature, their characteristics should be defined in statistical terms resulting from probabilistic analysis.

6 - Complex variables such as the characteristics of the pavement, the temperature conditions, the hydraulic conditions and the physical and mechanical responses of the pavement to variable temperature, hydraulic and other conditions should be defined by a systematic approach.

7 - A reliable system for the prediction of low temperature cracking of asphalt pavements should include the following sub-systems.
  - Layered sub-system
  - Temperature sub-system
  - Hydraulic sub-system
  - Physical and mechanical sub-system.

  A brief description of each sub-system is given henceforth.

2 - TYPICAL MULTILAYERED SUB-SYSTEM OF A FLEXIBLE PAVEMENT

Figure 4 presents a typical multilayered system of a first class flexible pavement for Canadian climate and also the typical geometric standards that go with it. The system is typically composed of 2 or 3 layers.

- An asphalt layer made up of one to 3 sub-layers of densely graded bituminous mixture for surface, base and foundation. The total thickness of this layer varies generally between 5 and 20 cm.
- A granular base layer either in crushed stone, in crushed gravel or in natural gravel. The thickness of this layer varies generally from 20 to 40 cm. The required characteristics of this layer depend largely on

the thickness of the asphalt layer. In most cases the specified granular materials are densely graded.

- A subgrade soil which can vary from clay to gravel.
- A 30 to 60 cm sandy subbase layer over very cohesive soil when necessary.

The characteristics and thickness of each layer varies within a certain range but when the subgrade soil is frost susceptible, most highway agencies will specify that the first 90 to 120 cm from the surface be made of non frost susceptible materials.

3 - TEMPERATURE SUB-SYSTEM

The most important single factor acting on the physical and mechanical behavior of pavement is the temperature.

The temperature regime of a pavement is influenced by geography, local weather, and exposure to sun, wind and other adverse conditions:

- Geography will determine the duration of day and night in winter and the importance of temperature cycling;
- Local weather will determine the lowest air temperature, the length of the cold spell and the rapidity of the temperature change;
- Exposure to solar radiation will determine the maximum daily surface temperature of the pavement although exposure to wind or adverse conditions (e.g. snow accumulation area or absence of snow locally) will be preponderant in the daily temperature distribution in the profile and at the surface of the road during a winter day.

Roads that are located in different weather environments will be influenced in different ways and the basic model cannot be generalised without taking into account the basic differences in winter conditions. For example, Table 1 presents typical winter conditions for Montreal and the James Bay area (14). There are many differences in the duration of day or night, duration and intensity of cold spell, solar exposure and so on.

In relation to the physical and mechanical behavior of a pavement, the temperature regime of a road can be divided into three (3) zones as illustrated in figure 5 :

- Zone A or daily temperature variation zone
- Zone B or intermediate temperature zone

- Zone C or seasonal temperature variation zone.

Zone A

The thickness of this zone varies between 30 to 50 cm depending on the thickness of the asphalt layer. Zone A is characterized by:

a) an immediate and rapid response to the variation of the surface temperature
b) a cyclic daily temperature variation
c) the highest absolute sub-zero temperature (θ) in the pavement which normally varies between 10°C at the base of the zone to 30°C at the surface of the asphalt layer
d) a great seasonal variation in temperature from 50°C to -40°C
e) a great variation in the rate of temperature changes; up to 6°C per hour.

All these characteristics have an influence on the properties of the material and the nature of stresses in the pavement (i.e. magnitude of stress and loading time).

Zone B

The thickness of this sandwich zone may vary between 40 to 80 cm and is characterized by:

a) a slow response to the variation of surface temperature, with a time lag varying from a day to a week or more, so that a reverse thermal gradient could occur in this zone
b) a mean minimum temperature which usually varies between -3 and -15°C
c) a smaller variation in the rate of temperature change between 0.1 to 0.5°C per hour
d) a smaller change in temperature with surface temperature; typically a surface variation of one degree produces a change of 0.1 degree in the layer
e) the absolute sub zero temperature ranging between 3 and 10°C.

Zone C

The thickness of this zone may vary between 100 to 400 cm depending on the depth of frost penetration. Zone C is characterized by:

a) An important time lag in the response to the variation of surface temperature. Time lag is normally more than a week.

b) A very low variation in daily temperature.

c) A mean temperature which usually varies little between 0 and -3°C, the minimum temperature in this zone being observed at the end of the cold season.

d) A very low rate of temperature variation which varies between 0.01 and 0.05°C/hour.

Snow cover area

Along road sides there are areas were the frost penetration and temperature changes are greatly influenced by snow cover. The thermal states of materials under snow cover are similar to those of Zone B and C.

4 - HYDRAULIC SUB-SYSTEM

A simplified model of the hydraulic sub-system is presented in figure 6. In relation to the physical and mechanical behavior of a pavement, the influence of water should be examined in 3 different phases :

- Water in the unfrozen state
- Water upon freezing
- Water in the frozen state.

The depth of the water table and so also the pre-winter water distribution in the pavement structure depend mainly on the geometry and geography and the properties and arrangement of layered materials in the pavement. The main properties which affect water in the unfrozen state are permeability, residual saturation or effective porosity and absolute porosity. Typical hydraulic properties of pavement layers are summarized in table 2. Note that a change in the water content of soils and paving material can affect the structural capacity of the pavement and can, in some plastic soils, cause shrinkage or swelling.

Except in very few cases, upon freezing, moisture is attracted toward the cold front and so will increase the moisture content in the pavement (19-23). The attraction will depend on the initial saturation of the material and on the rate of frost penetration. Moisture change in the pavement may occur under two different freezing conditions generally called the closed system (without water supply) and the open system (with water supply). In most studies the open system is used. Water saturated materials are more representative of conditions inside the pavement (25-26).

In a closed system, most materials will have an increase of saturation in the upper layer and will dry in the lower layer. The presence of an anti-capillary sandy subbase with low residual saturation under a densely graded base course might change these conditions: as the suction of graded material is higher than sand, water in the sand will be attracted by the dense base layer and increases its degree of saturation. This will increase the temperature gradient at the boundary of the two materials as the temperature will become lower in the more conductive densely graded layer. If the capillarity of the sand layer is low, cracks might form in the layer as a result of the drying shrinkage and that might affect the behavior of the pavement.

In the frozen state, very coarse material at low saturation or very fine material will be affected at temperatures just below the freezing point. With coarse granular material, water vapor diffusion and sublimation at the top of the layer may occur.

Vapor diffusion is a main concern in a material in which saturation is below 30%, as its thermal conductivity will decrease with decreasing water content. Sublimation on the other hand might influence the mechanical properties of granular material as it may then behave as an unfrozen material.

With very fine material, unfrozen water content will be high and will give frozen materials a high permeability which might facilitate water transfer.

5 - PHYSICAL AND MECHANICAL RESPONSES SUB-SYSTEM

As show in figure 3, physical and mechanical behavior of the pavement and ultimately its low temperature cracking is the result of the combined effects of:

- The characteristics of the pavement defined by the layer sub-system
- The hydraulic conditions defined by the hydraulic sub-system
- The temperature conditions defined by the temperature sub-system and also the effects of other factors not discussed in this paper, such as fatigue under repeated loading, heaving by frost action, settlement of

compressive soil, swelling and shrinkage of very plastic soils.

The following responses are of great importance to low temperature behavior of the multi-layered pavement:

- those affecting the volume change of material upon freezing and when frozen
- those creating stresses in the multi-layered system, either at the interfaces or within the layers
- those affecting the strength of materials in the system.

Figure 7 presents the principal elements of the sub-system responses: it underlines the fact that if the behavior of the pavement is to be evaluated with a dependable precision, there must be a reasonable agreement between the behavior of materials evaluated by laboratory testing and the actual behavior of materials in the global system.

5.1 - Volume changes under different climatic conditions

Volume change in a bituminous mixture is related to its coefficient of expansion. Although the volume change is different above and below the glass transition point (26), many authors do not consider it as temperature dependant.

Volume change of granular material depends on the state of the water in the material. The volume change curve can as illustrated in figure 8 be divided into 3 distinct parts:

- Temperature dependant shrinkage above 0°C in the non frozen state.
- Ice forming expansion between 0 and -5°C upon freezing.
- Temperature dependant shrinkage below -5°C at the frozen state.

The transition temperature between part 2 and part 3 varies between -1°C for highly compacted granular materials to -5°C for sandy clay.

Upon freezing, the linear variation is high, e.g. from 0.1% to 0.8% for granular base material depending on the degree of saturation and the percent of fines.

Volume change of clay materials depends mainly on the presence of expansive clay minerals and the degree of saturation (12, 27). If clay contain expansive clay minerals and if the degree of saturation is below 70% the clay will shrink between 0 and -5°C. Now, if the degree of saturation is above 70%, the volume change pattern is similar to that of granular material: expansion instead of shrinkage occurs between 0 and -5°C (see figure 9).

5.2 - Mechanical behavior under different climatic conditions

The mechanical behavior of bituminous mixtures is temperature and loading time dependant. Generally speaking the stiffness of mixtures varies with the stiffness of asphalt and increases with decreasing temperature. On the other hand the tensile strength also increases with decreasing temperature with a maximum around -15°C, although most of the tests have been done at very fast rate of loading compared to the temperature stress building rate. Some experiments (24, 25) show that asphalt has some ductility at low temperatures and low rates of loading, and that it might have great influence on cracking. Note that very little work have been done to determine the effect of water in different states on the mechanical behavior of asphaltic mixtures.

Typical mechanical behavior of frozen granular material is illustrated on figure 10. As indicated in the figure the strength of a frozen granular material depends mainly on temperature and loading rate, and the strain on the loading time.

As indicated in figure 10a at very low loading rates the materials have a ductile behavior with compressive strength equal to tensile strength (friction angle $\phi = 0$). At fast loading rate, the material become brittle and the compressive strength will be 4 to 6 times higher than the tensile strength and ϕ almost equals the unfrozen friction angle (26, 27). Note that the ductility point for a given loading rate varies with temperature and ice saturation.

Figure 10b gives a typical relationship between temperature and strength for a medium loading rate above the ductility point. It shows that both tensile and compressive strength of granular material will increase with the absolute subzero temperature.

Figure 10c illustrates the influence of loading time and strain for different stresses. It shows that below the critical stress, which depends on water content and temperature, frozen granular materials have an elastic behavior while at higher stresses, the material is visco-elastic and creeps under stress.

Based on our recent lab and field test results, it can be said that the critical stress of densely graded base course material varies typically from 1000 Kpa at $-2^{o}C$ to 4500 Kpa at $-6^{o}C$.

Typical mechanical behavior of clay and sandy materials is illustrated in figure 11. As indicated the behavior is mainly influenced by ice saturation, temperature and the rate of loading.

Figure 11a shows that the shape of the strength loading rate relationship is similar to frozen granular material.

Figure 11b illustrates the importance of ice saturation on the strain time relationship. At very low ice saturation the material is highly deformable at high or low stress. At intermediate ice saturation the material is slightly deformable at low stress whereas at high stress the material is much more deformable even at high ice saturation. Creep deformation is so important for silt and clay that many authors suggest that elastic and plastic deformation can be neglected at temperatures ranging between 0 and $-5^{o}C$ and if loading time exceeds 24 hours (29, 30).

Figure 11c illustrates the strength time curves of frozen sand and clay. It shows that frozen materials have residual strength (which is the long term stress that could be applied without rupturing the material by creep deformation). Typically the residual strength of clay (C, in Kpa) or the safe shear stress varies with absolute subzero temperature (T) between 0 and $3^{o}C$ (28) according to the equation:

$$C = 35 + 28\ T$$

5.3 - Mechanical behavior at the layers boundaries

Each layer of the layered system if looked at separately will have a particular mechanical behavior, as the material, the temperature conditions, and the hydraulic conditions are different as well as the possibility of interacting and being influenced by the behavior of adjoining layers. Each layer will be submitted to a stress and strain condition related to a particular temperature and loading condition and will either receive or transfer part or all of the stresses to a more rigid (i.e. cooler) layer above. Usually materials are more rigid at the surface and somewhat viscous near the freezing front. It has been shown (31) that stress transfer from a viscous material to a stiffer material could cause or contribute to cracking.

There are other considerations too. The boundary condition of each layer has a dominant effect on stress. As indicated by Ahlboro and Poehlman (32), the instantaneous adhesion between ice and asphalt or other granular materails is high but releases with time because of creep effects. So, the interlayer adhesion will be strong when the stress is first applied but, as the adhesion gives way, there will be a shear stress transfer changing the whole stress pattern in both layers under consideration.

5.4 - Effect of traffic load

The behavior of a frozen material is highly time dependant: fast traffic will induce high strength, whereas slow traffic and idle vehicles will induce creep which can eventually lead to cracking. In some cases the traffic load will be added to the already present stress resulting from the hydraulic and temperature states and could cause additional damage to the pavement.

6 - FRAMEWORK OF A MODEL FOR THE PREDICTION OF LOW TEMPERATURE CRACKING

The proposed framework of a global predictive model is illustrated in figure 12. It consists of:

1) defining the significant parameters needed:
    a) to characterize the composition of the layered system and the properties of each individual layer (i.e. parameters used in the layer sub-system)
    b) to characterize the hydraulic conditions in the non frozen, upon freezing and frozen states (i.e. parameters used in the hydraulic sub-system)

   c) to characterize the temperatures in the 3 distinct zones described before (i.e. parameters used in the temperature sub-system)
      A list of the principal parameters involved in the three subsystems is given in table 3.

2) develop a series of models to predict the physical and mechanical behavior of each layer in relation to other layers. These models can be divided into 4 groups as follows:
   a) Models for predicting the stresses and deformation of each layer caused by shrinkage, expansion, hydraulic pressure or external loads, etc.
   b) Models for predicting the weakening of the system by stress concentration and propagation, warping, creep, age hardening of BC etc.
   c) Models for predicting the strength of each layer and the combined effects.
   d) Models for predicting the decrease in strength by thermal fatigue, fatigue under external loads, ice segregation and ice in cracks, interlayer adhesion, etc.
3) develop an integrated model to predict the probability of cracking.
   A resume of the principal mechanisms which could be involved in the cracking phenomena, either alone or in combination is given in table 4: each mechanism can involve one particular layer or several layers of the structure. For example:
   - Rupture of the asphalt layer could be influenced by thermal shock, fatigue cracking or stress transfer phenomena.
   - Rupture of the granular base and subbase can occur either when freezing (shrinkage drying before and after the freezing time) or when frozen (fatigue cracking in the zone A or creep deformation with stress transfer or concentration).
   - Subgrade materials will mainly crack by hydraulic fracturing during the freezing period or by creep deformation leading to either stress concentration in the upper layers or deep seated cracking (involving shoulders and open frost cracking in northern regions). Transition zones in rock cuts will be the sites of thermal disturbance and might affect the behavior of nearby subgrade material.

As time has an important effect in many mechanisms of cracking (creep and fatigue cracking) and it influences the total depth of frost penetration, the occurrence of cracking, defined as the maximum distance between cracks, will be time dependant.

Finally, as stated earlier, most of the design parameters are variables in nature. As some of the input data have had to be evaluated on the basis of probability, the final results on the type, number and distance of cracks could only have been evaluated by statistical and probabilistic analysis and expressed in probabilistic terms.

7 - CONCLUSION

The behavior of a pavement structure submitted to freezing and thawing is the result of complex phenomena related (a) to the existence of many systems acting on the structure: temperature, hydraulic, physical and mechanical characteristics of different layers and (b) to the interaction of different layers. The understanding of all the possible actions and interactions along with the use of appropriate analytical techniques will give the required insight into the design of pavements to minimize low temperature cracking.

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Table 1: Comparison between Montreal and La Grande area Thermal Conditions

	Montreal area	La Grande area
LATITUDE	45.5	52.5
Length of day Dec. 21th Jan. 21th	 8.53 hr 9.27 hr	 8.04 hr 8.33 hr
Winter mean Temperature	-5°C	-20°C
Solar altitude Dec. 21th	22°C	14°C
Insolation time Winter Summer	 100-125 hr 800 hr	 60-80 hr 780 hr
Normal number of days below - 23°C - 29°C - 35°C	 0,2 0,0 0,0	 4,2 (max 9) 2,3 (" 6) 0,6 (" 3)
Freezing - degree-day mean maximum	 1400 (σ = 323) 2100	 4700 (σ = 700) 5600
Number of days below 0°C	208	240
Degree-day, first month	326 (max = 539)	175 (max = 300)
Daily temperature variation under snow cover	nil	2-3°C

Based on data from reference 14

Table 2: Hydraulic properties of pavement materials currently used in Quebec

Material	Water content (1)	Residual Saturation (2)	Effective porosity (3)	Normal permeability (4)
Asphaltic concrete	1-2%	-	-	E-10 to E-11
Base course materials	4-6%	0.6-0.7	.02-.05	E-05 to E-06
Subbase materials	6-12%	0.3-0.4	.20-.30	E-02 to E-04
Subgrade				
- granular soil	10-20%	0.6-0.99	.01-.30	more than E-06
- cohesive soil	25%	= 1.00+	.01-	less than E-06

Notes:

(1) Water content is in percent by weight

(2) Residual saturation in decimal of pore space, determined after a long drainage period

(3) Effective porosity is in decimal, it is the volume of water that could be drained by gravity

(4) Permeability is in centimetres per second. E-06 means 10 to the exponent -6

Table 3 : Factors influencing the frozen behavior of the pavement

Temperature conditions	
At the surface	- Intensity and duration of cold spell - Minimum and maximum variation of temperature - Duration of day and night - Exposition to sun, wind and snow accumulation - Albedo of the surface and its cleanliness
In depth	- Thermal properties of materials: conductivity, specific heat, latent heat etc. - Rate and range of temperature variation - Thermal gradient and depth to the freezing front - Possibility of temperature inversion
Moisture conditions	
Before freezing	- Weather regime in the period preceeding frost time = rain and snow accumulation and frequency - Geometry and state of the pavement surface - Depth to the water table
Upon freezing	- Permeability and saturation of the materials - Water suction or water expulsion potential
Physical Characteristics	
Asphalt layer	- Thickness - Type of mix: penetration and softening point of asphalt, asphalt contents, voids content, volume concentration of aggregate - Nature and grading of aggregates - Nature of filler
Granular layer	- Thickness and position in the pavement structure - Compactness and gradation - Percentage of fines and binding materials - Composition and plasticity of fines - Presence of porous and deleterious materials
Subgrade material	- Geometric arrangement and thickness related to depth of freezing front - Nature, composition and gradation of materials - Compactness - Nature and plasticity of fines: presence of deleterious materials, of expansive clay minerals, etc...
Mechanical characteristics	
All layer, per se	- Stiffness or viscosity, absolute and relative - Ductility-brittleness transition point - Critical stress or strength at different temperatures and rates of loading - Unfrozen friction angle and cohesion - Compressibility and poisson ratio
Inter layer behavior	- Adhesion between layers, short term and residual - Nature and magnitude of stress transfer between layers

Table 4 : Principles factors contributing to low temperature cracking of asphalt pavement

1- Upon Freezing	
Shrinkage or expansion (in zone A)	Cracking of a layer will result when the tensile stress caused by shrinkage and/or expansion is greater than the tensile strength of layer.
Hydraulic fracturing (in Zone B and C)	Upon freezing, water in granular materials could be expelled into an adjacent non frozen layer. Cracking in this layer will result when the hydraulic pressure created is higher than its tensile strength.
Shrinkage by freeze drying (in zones A & B)	Partial drying of a clay or sandy layer may cause cracking either by sublimation or transfer of water towards the freezing front, if water cannot be available from a lower layer.
2- When Frozen	
Thermal Shock (in zone A)	Rapid changes in temperature at the surface resulting from snow removal or rapid warming or cooling of the surface will create a temperature differential which may lead to stress concentrations by transfering stresses from the softer layer to the stiffer layer, and, ultimatly may lead to the rupture of the asphalt layer.
Thermal Fatigue (in zone A)	Thermal expansion and contraction associated (or not) with tensile strain caused by traffic will induce fatigue which may lead to cracking.
Relaxation of interlayer adhesion (between zone A & B)	Adhesion between layers allows stress transfer and produce a particular stress pattern in the system. Relaxation with time will redistribute the type of stress from a bending stress to a purely tensile stress and so will affect the cracking.
Water filling of existing cracks (in zone A & B)	Water filling of existing cracks, either macro or microscopic, will change the mechanical behavior of the frozen material from elastic behavior to highly viscous behavior.
Creep (in zone A, B and C)	Cracks migth result under long term temperature induced stress in zones B & C and also in zone A when the stresses are higher than the strength.

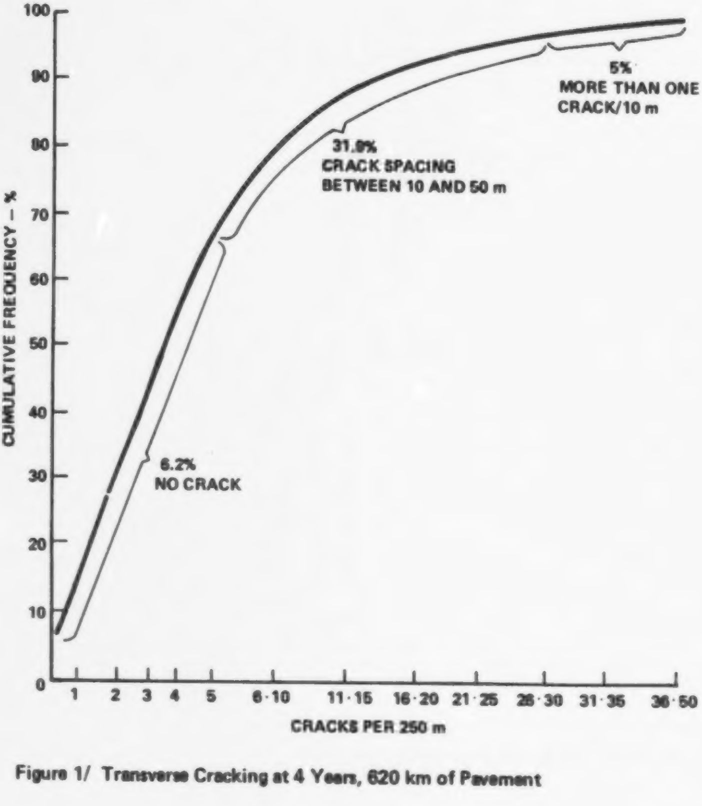


Figure 1/ Transverse Cracking at 4 Years, 620 km of Pavement

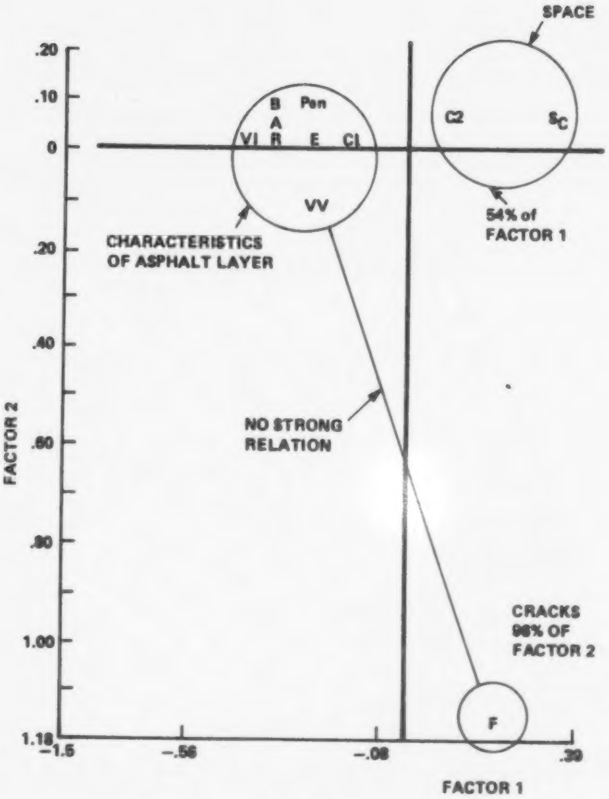


Figure 2/ Results of Correspondence Analysis

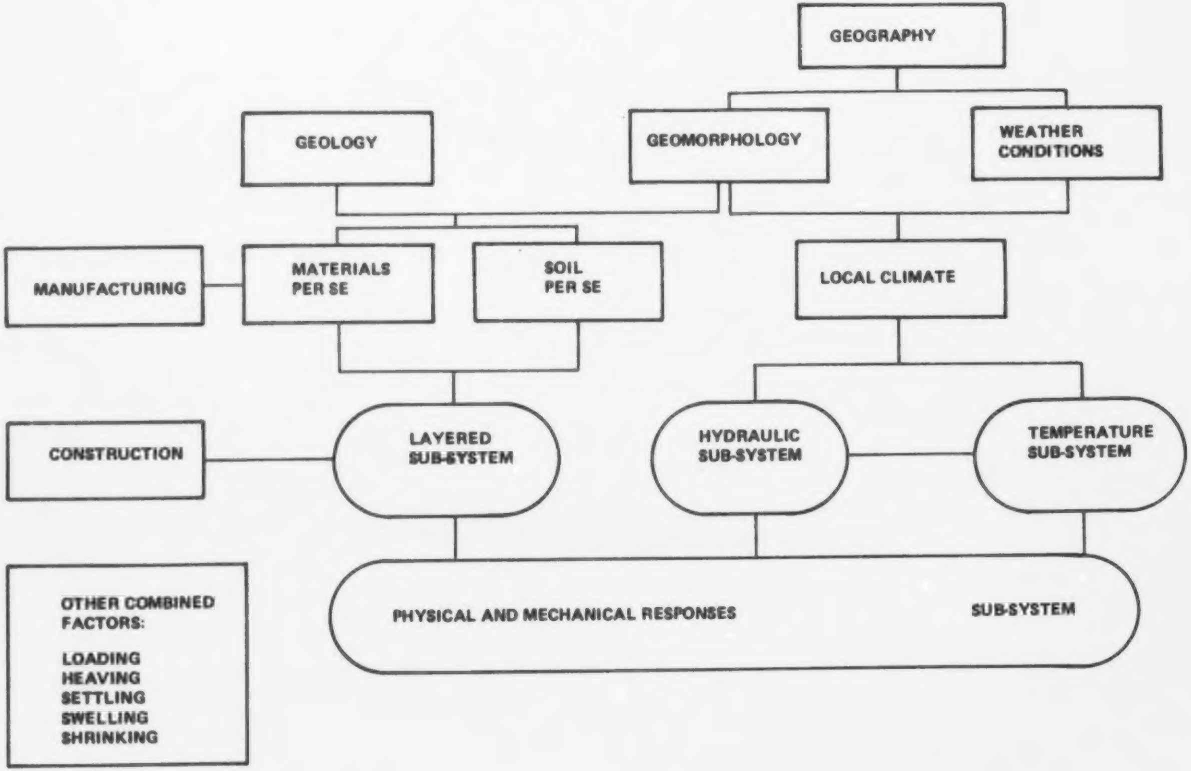

GEOGRAPHY
GEOLOGY
GEOMORPHOLOGY
WEATHER CONDITIONS
MANUFACTURING
MATERIALS PER SE
SOIL PER SE
LOCAL CLIMATE
CONSTRUCTION
LAYERED SUB-SYSTEM
HYDRAULIC SUB-SYSTEM
TEMPERATURE SUB-SYSTEM
PHYSICAL AND MECHANICAL RESPONSES SUB-SYSTEM
OTHER COMBINED FACTORS:
LOADING
HEAVING
SETTLING
SWELLING
SHRINKING

Figure 3/ Concept of the Global Approach

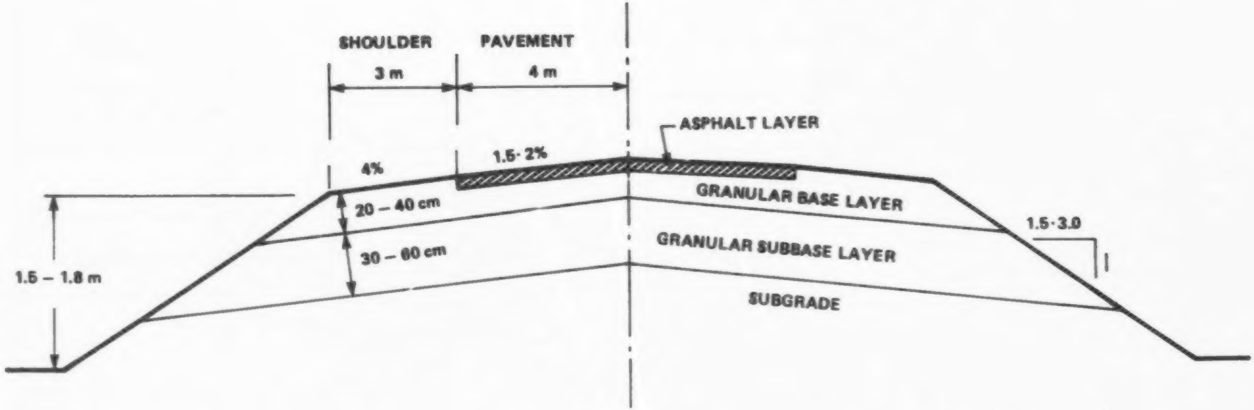

SHOULDER
PAVEMENT
3 m
4 m
ASPHALT LAYER
4%
1.5 · 2%
20 – 40 cm
30 – 60 cm
1.5 – 1.8 m
GRANULAR BASE LAYER
GRANULAR SUBBASE LAYER
SUBGRADE
1.5 · 3.0
1

Figure 4/ Typical Multilayered System of a Flexible Pavement for a First-Class Road

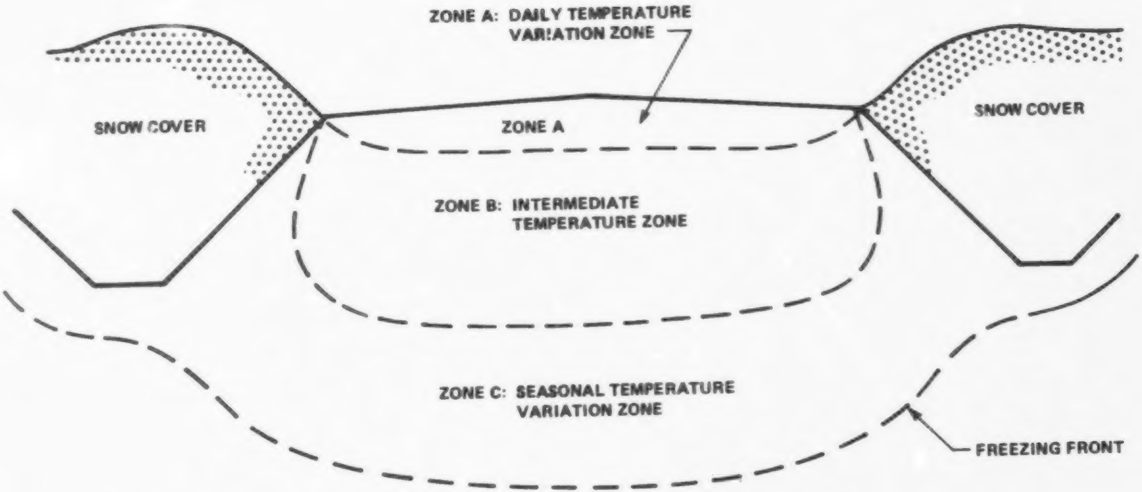

ZONE A: DAILY TEMPERATURE
VARIATION ZONE
SNOW COVER
ZONE A
SNOW COVER
ZONE B: INTERMEDIATE
TEMPERATURE ZONE
ZONE C: SEASONAL TEMPERATURE
VARIATION ZONE
FREEZING FRONT

Figure 5/ Temperature Zoning of a Pavement

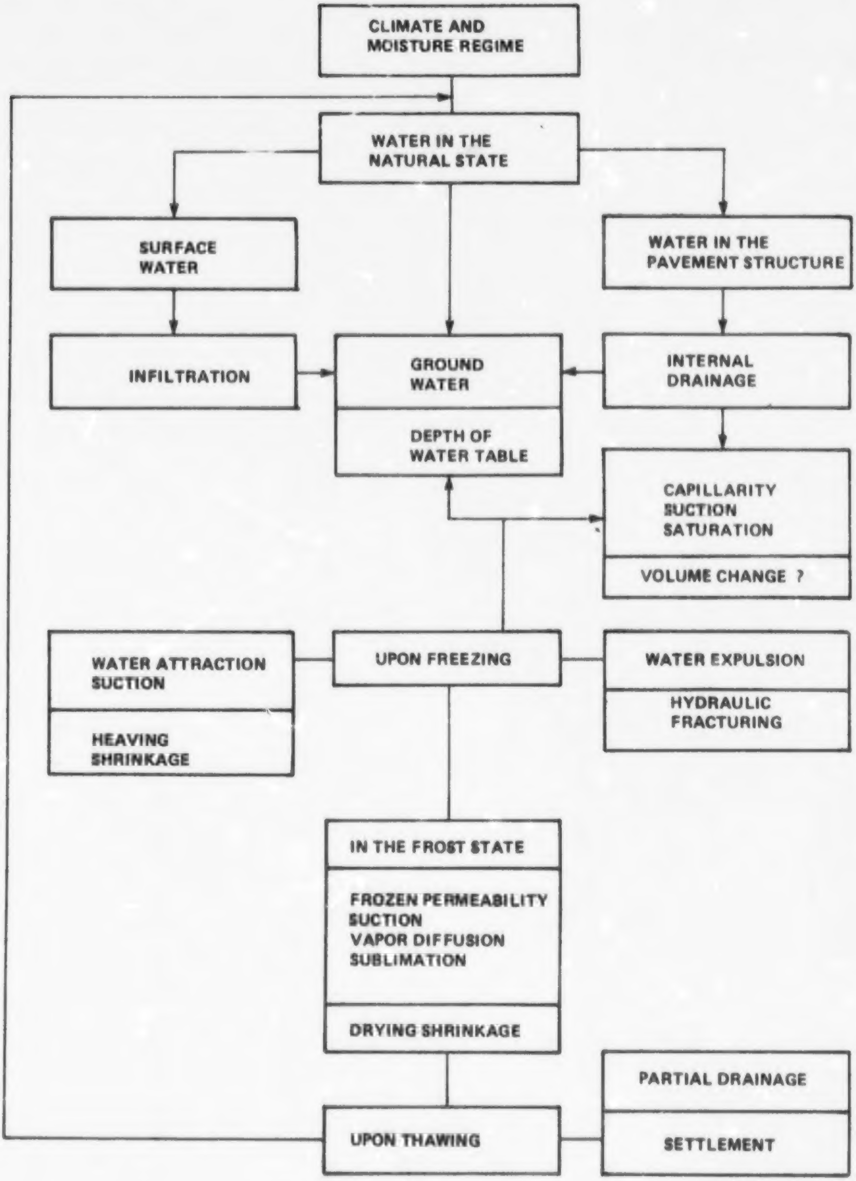

CLIMATE AND
MOISTURE REGIME
WATER IN THE
NATURAL STATE
SURFACE
WATER
WATER IN THE
PAVEMENT STRUCTURE
INFILTRATION
GROUND
WATER
DEPTH OF
WATER TABLE
INTERNAL
DRAINAGE
CAPILLARITY
SUCTION
SATURATION
VOLUME CHANGE ?
WATER ATTRACTION
SUCTION
HEAVING
SHRINKAGE
UPON FREEZING
WATER EXPULSION
HYDRAULIC
FRACTURING
IN THE FROST STATE
FROZEN PERMEABILITY
SUCTION
VAPOR DIFFUSION
SUBLIMATION
DRYING SHRINKAGE
PARTIAL DRAINAGE
SETTLEMENT
UPON THAWING

Figure 6/ Simplified Hydraulic Sub-System

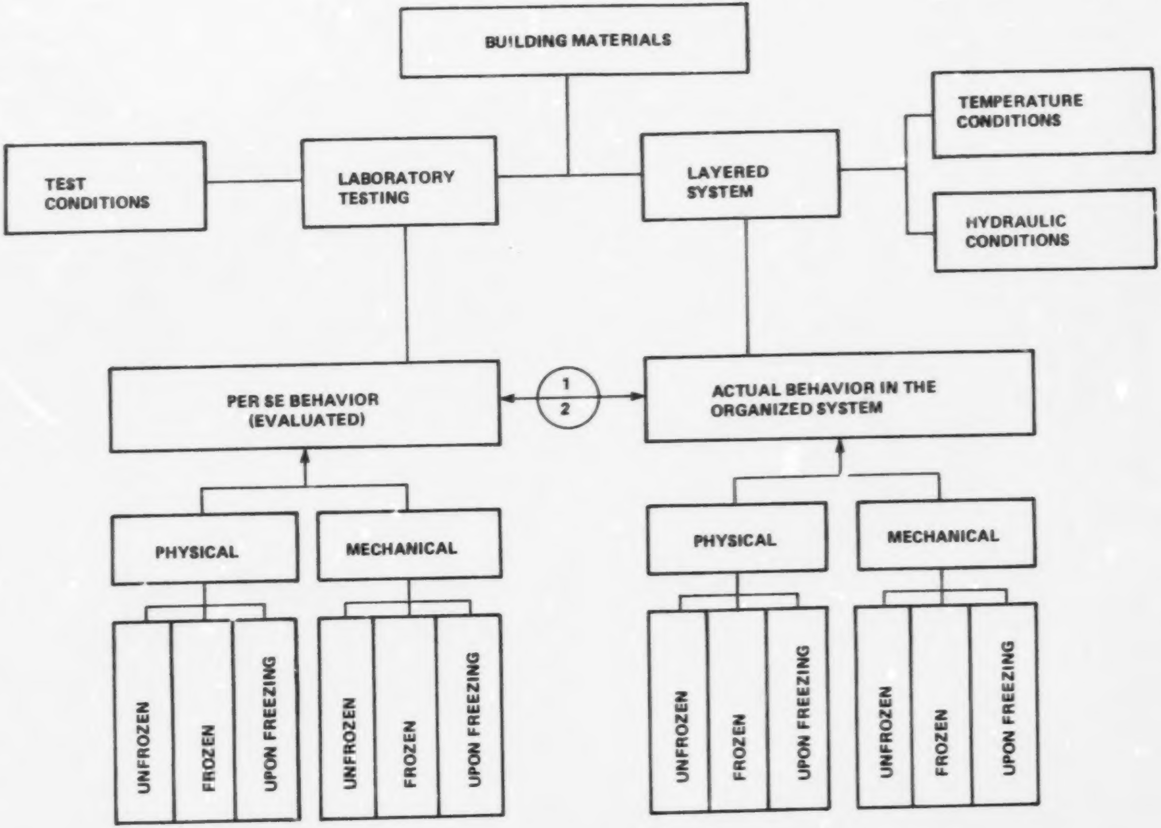

BUILDING MATERIALS
TEST CONDITIONS
LABORATORY TESTING
LAYERED SYSTEM
TEMPERATURE CONDITIONS
HYDRAULIC CONDITIONS
PER SE BEHAVIOR (EVALUATED)
1
2
ACTUAL BEHAVIOR IN THE ORGANIZED SYSTEM
PHYSICAL
MECHANICAL
PHYSICAL
MECHANICAL
UNFROZEN
FROZEN
UPON FREEZING
UNFROZEN
FROZEN
UPON FREEZING
UNFROZEN
FROZEN
UPON FREEZING
UNFROZEN
FROZEN
UPON FREEZING

Figure 7/ Physical System Affecting the Pavement Structural Behavior

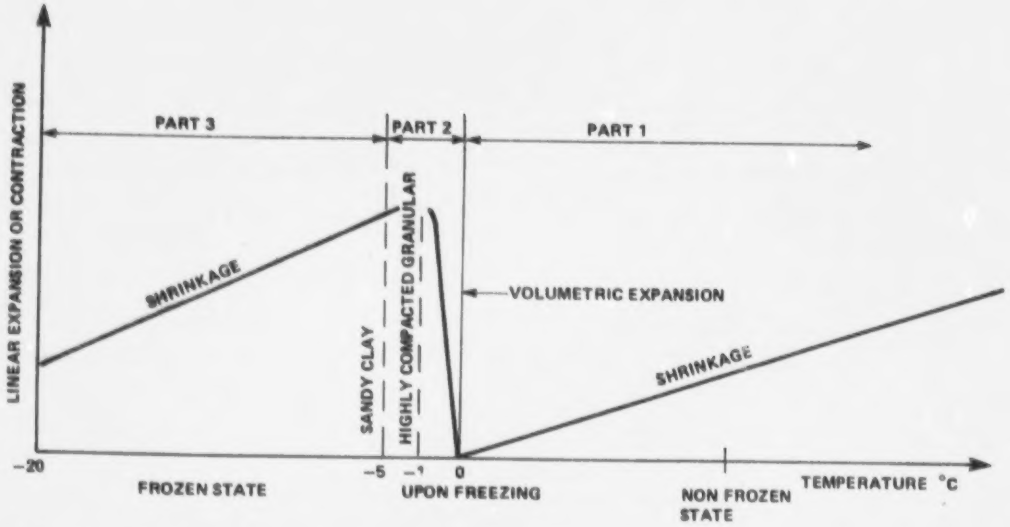
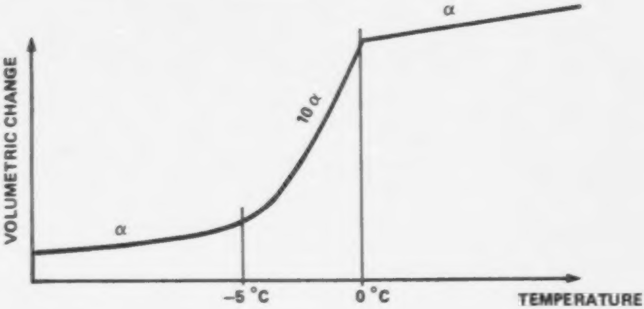


Figure 8/ Typical Volume Change Curve for Granular Material.

α: COEFFICIENT OF THERMAL EXPANSION

a) DEGREE OF SATURATION BELOW 70%



b) DEGREE OF SATURATION ABOVE 90%

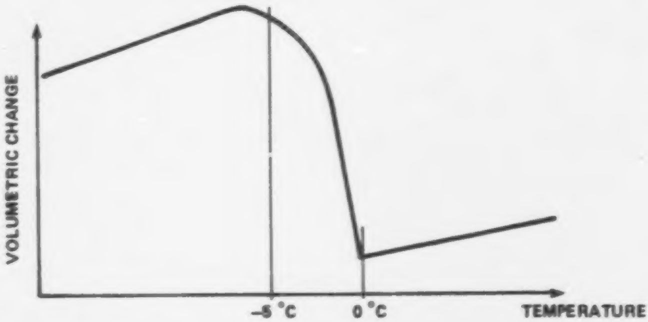
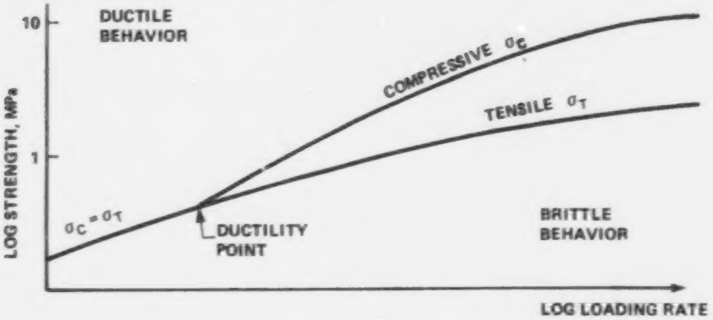
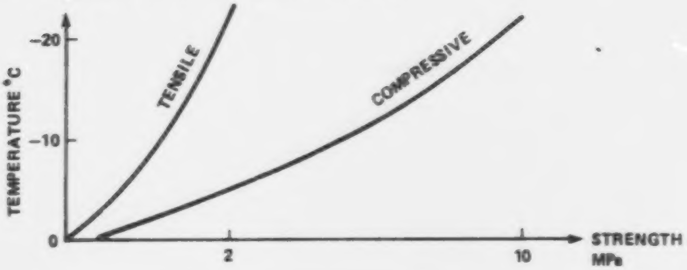


Figure 9/ Typical Volume Change Curve for Clay Material Containing Expansive Material

a) STRENGTH/LOADING RATE RELATIONSHIP



b) STRENGTH/TEMPERATURE RELATIONSHIP



c) STRAIN/TIME RELATIONSHIP

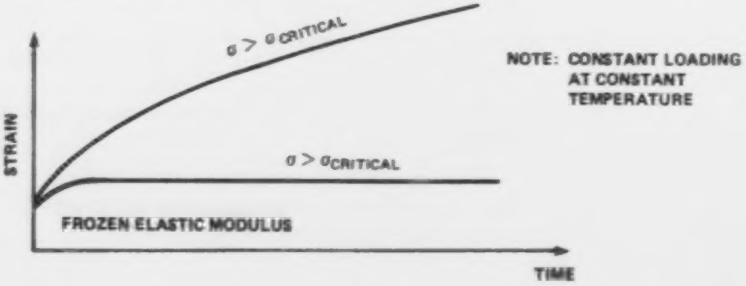
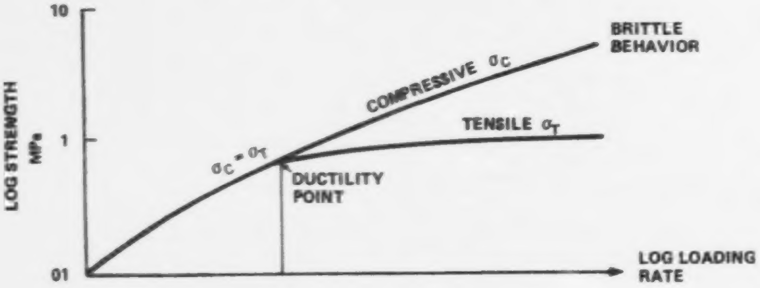
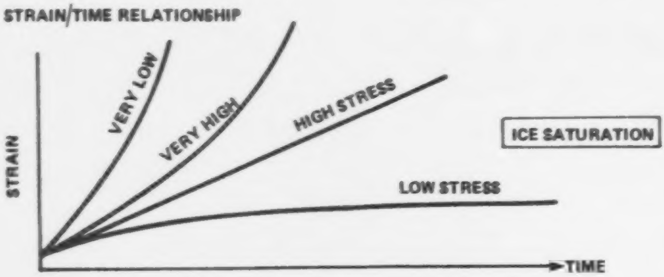


Figure 10/ Typical Mechanical Behavior of Frozen Granular Materials

a) STRENGTH/LOADING RATE RELATIONSHIP



b) STRAIN/TIME RELATIONSHIP



c) STRENGTH/TIME RELATIONSHIP

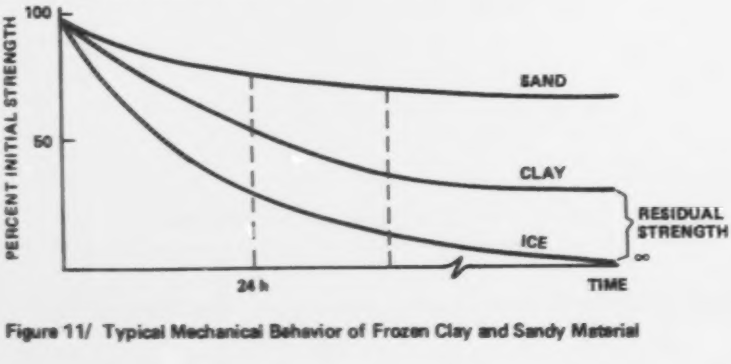


Figure 11/ Typical Mechanical Behavior of Frozen Clay and Sandy Material

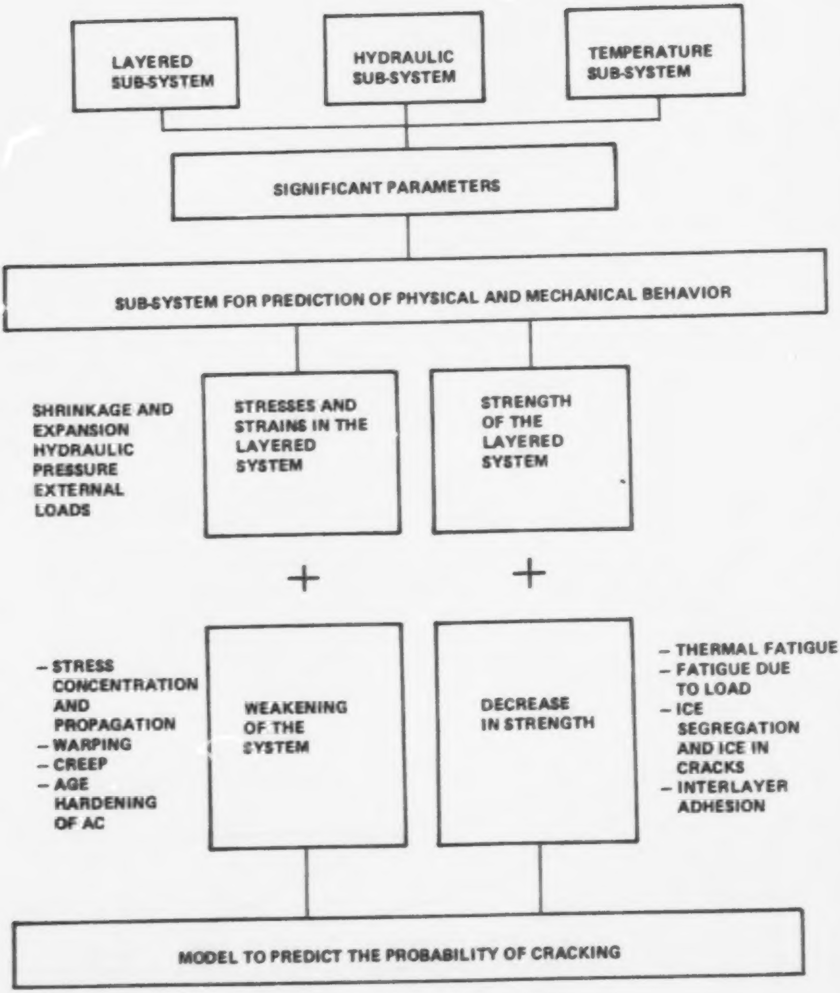

LAYERED SUB-SYSTEM
HYDRAULIC SUB-SYSTEM
TEMPERATURE SUB-SYSTEM
SIGNIFICANT PARAMETERS
SUB-SYSTEM FOR PREDICTION OF PHYSICAL AND MECHANICAL BEHAVIOR
SHRINKAGE AND EXPANSION HYDRAULIC PRESSURE EXTERNAL LOADS
STRESSES AND STRAINS IN THE LAYERED SYSTEM
STRENGTH OF THE LAYERED SYSTEM
+
+
– STRESS CONCENTRATION AND PROPAGATION
– WARPING
– CREEP
– AGE HARDENING OF AC
WEAKENING OF THE SYSTEM
DECREASE IN STRENGTH
– THERMAL FATIGUE
– FATIGUE DUE TO LOAD
– ICE SEGREGATION AND ICE IN CRACKS
– INTERLAYER ADHESION
MODEL TO PREDICT THE PROBABILITY OF CRACKING

Figure 12/ General Framework of a Model for Predicting Low Temperature Cracking

Paving
in Cold
Areas

Vancouver B.C., 1982

Session 1

Use of Field Observations and Distress Prediction Techniques to Improve Low Temperature Pavement Cracking Performance

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The University of Alberta
Edmonton, Alberta, Canada

USE OF FIELD OBSERVATIONS AND DISTRESS PREDICTION TECHNIQUES TO IMPROVE LOW TEMPERATURE PAVEMENT CRACKING PERFORMANCE

by
K.O. Anderson
Professor of Civil Engineering
The University of Alberta

Introduction

Concern over the problems associated with low temperature cracking of asphalt pavements has been reflected by numerous research activities in Canada and the USA particularly since the early 1960's. Early reports published at that time (1,2) were supplemented greatly when a Symposium on Non-Load Associated Cracking was sponsored by the Association of Asphalt Paving Technologists in 1966 (3). Comprehensive reviews of the literature have been published periodically since then (4,5,6,7), however it is not possible to further summarize this vast amount of information adequately in the time and space available at this Workshop. This paper will be of limited scope with the intention of treating some of the studies in which the author has had some involvement over this period of time. Extensive work of other researchers, such as McLeod and Haas, is recognized but will not be discussed in this paper since their own contributions may be forthcoming.

Most of the report to follow is based upon investigations on the rural highway system of Alberta, or on data made available from the Ste. Anne Test Road in Manitoba (8-15). Studies which involve the development of distress prediction models based on this experience such as the COLD program (16), along with others that help to verify or calibrate the model (17-19) will also be briefly described. Work in Alberta was undertaken within the framework of the Alberta Cooperative Highway Research Program, with some support of the later verification phase under an operating grant of the National Research Council of Canada. The damage prediction model referred to as COLD (Computation of Low-Temperature Damage) was developed under the National Cooperative Highway Research Program Project 1-10B by Woodward-Clyde Consultants, San Francisco. The research in Texas was under a cooperative research program through the Texas Transportation Institute at The Texas A & M University, College Station.

Background

In the early studies concerning the performance of asphaltic concrete pavements subjected to the severe climatic conditions of western Canada it was evident that the asphalt cement used was a significant factor. Extensive surveys of the main highway system had generally indicated that the larger amounts of transverse cracking appeared associated with certain asphalt sources (1,8). Cracking frequencies were as high as 450 per mile (280 per km), for an average minimum interval of 12 feet (3.6 m). Asphalt cements of 200-300 penetration grade were commonly used, although some 150-200 grades were used on occasion.

Severe performance losses occurred on pavements that were built on swelling clay type subgrades and had ineffectively maintained transverse cracks. One particular section of pavement required remedial action in less than 4 years due primarily to this phenomena.

Based on extensive surveys of crack occurrence on main highways and early results from a test section constructed under particularly close observation and construction control (9), the Department of Highways of Alberta revised their asphalt cement specification in 1967. This revised specification called for a minimum penetration of 250, at 25°C (77°F), with a minimum absolute viscosity of 275 poises at 60°C (140°F) for material supplied. Experience with this specification which restricted the use of "low viscosity" asphalt cements at a given penetration, resulted in a more uniform material being supplied from the various local sources. As well there was a reduction in the occurrence of abnormally high transverse crack frequencies (11). These had previously been encountered with "low viscosity" penetration graded asphalt cements. Despite the asphalt cement being slightly softer at most temperatures, there did not appear to be any greater permanent deformation in service than heretofore (15).

It was noted that coincident with this change in asphalt specification in 1967, a concerted effort was implemented to compact plastic subgrade soils closer to and possibly, slightly above optimum water content. This was in contrast to earlier practice where much compaction was carried out at a low degree of saturation leading to high swelling potential at the cracks. Therefore, the often dramatic and early loss in ride quality has been reduced substantially even where high cracking frequency has been observed (15,20).

Systematic representative sampling and testing of asphalt cements used on Alberta highway projects was begun in 1964, in order to initiate the development of a data bank which could hopefully yield information relating the rheologic properties to service behavior. Asphalt cements as supplied, after accelerated hardening with the Thin Film Oven Test and/or Rolling Thin Film Oven Test, and recovered from cores taken from in service pavements periodically were tested for penetration, viscosity and softening point. Temperatures of 60°C, 25°C and 4°C were used for viscosity tests with some penetration tests at 0°C in addition to 25°C and 4°C.

In order to develop a suitable test method capable of testing asphalt concrete in tension at low temperatures, the tensile splitting test was investigated. A method of conducting the test on laboratory prepared specimens and cores capable of evaluating stress-strain characteristics to failure was described (21). Failure strain appeared to be a function of the asphalt cement, and that the occurrence of cracking increased as the failure strain decreased. Test temperatures were generally 0°F (-18°C), which was considered reasonably close to the temperatures at which cracking was likely to occur (9).

A significant development concerning stress prediction and thermal fracture susceptibility was made possible by the construction and observations on two test projects in Western Canada (9,10,12,14). The Manitoba test project, more commonly referred to as the Ste. Anne Test Road, was a joint research project of the Manitoba Department of Transportation and Shell Canada Limited. The primary variables were three pavement structures, four different asphalt cement binders, two asphalt contents and two aggregate gradations. Seven of the 29 pavement sections had crack detection circuits, used to define the initial times of cracking. Continuous measurements of air and pavement temperatures were recorded. The Alberta test project incorporated three different sources of asphalt cement of 200-300 penetration grade, representing low, intermediate and high viscosity materials as measured at 140°F (60°C). Crack detection and temperature measuring instrumentation was also continuously monitored.

Low temperature tensile fracture strengths of the asphalt concrete were determined by means of the tensile splitting test at a loading rate of 0.06 in./min (1.5 mm/min.) at temperatures ranging from the 20°F to -20°F (-7°C to -29°C). Stiffness modulus values for the asphalt concrete at 7200 seconds loading time and appropriate temperatures were estimated using Van der Poel's nomograph. As supplied asphalt cement properties were used in view of the premature hardening that was expected on the uncompacted paving mixtures which was expected to have affected the recovered properties.

A number of numerical methods were used to predict the thermal stresses in the asphalt concrete pavements based on elastic and viscoelastic analyses. Typical results of the thermally induced stress at the 1/2 in. depth (12.5 mm) are presented in Figure 1. Observed cracking in the test projects were compared to that predicted by the various stress analyses. Correlation was obtained between the observed and predicted fracture times. The simpler pseudo-elastic beam analysis was considered to yield reasonable results.

This work was used as a basis for the distress prediction model COLD, developed by Finn and project staff on NCHRP Project 1-10B (16). Modifications were made specifically to accommodate limited climatological data from weather reports and to provide for probability estimates based on variance associated with mix modulus and tensile strength.

Program COLD follows the flow diagram shown in Figure 2. It has been designed to a) compute the thermal regime of the asphalt concrete, b) compute the thermally induced tensile stress in the asphalt concrete, and c) compare the computed thermally induced stress with the measured tensile strength and indicate those conditions that could induce fracture. The information required for program COLD is as follows:

(a) Section thickness
(b) Thermal properties of component layers
(c) Ambient temperature and solar radiation
(d) Initial ambient temperature gradient
(e) Stiffness - temperature relationships of asphalt concrete
(f) Tensile strength - temperature relationship of asphalt concrete.

The modified program uses only the maximum and minimum air temperature values for each day of the study period, whereas Christison's detailed program used the temperature at each 2-hour time interval. With this modification, the climatological information on or near the location being studied can be used by referring to the monthly meteorological records. Verification of the simplified program is necessary in order to have confidence in the basic prediction method of COLD.

It should be recognized that this prediction method is intended to consider stresses induced by thermal shrinkage of the surface only. Transverse cracks can occur in the subgrade or base course materials due to changes in moisture and/or freezing temperatures and can propagate through the pavement structure and cause cracking in the surface (8,20,22).

Verification of the COLD Subsystem

The initial work of verification undertaken by this author was based on the original data from the Ste. Anne Test Road and a highway project in Alberta (17). The most recent work has involved a number of test sections in West Texas (18). Further development, verification and calibration has been conducted as an extension of NCHRP Project 1-10B using data obtained from the Utah Department of Transportation, by Woodward-Clyde Consultants (19).

Manitoba Test Project - Ste. Anne Test Road. Since the COLD program is intended to utilize records from the closest meteorological station, a comparison was made between the air temperature values recorded at the test site and the station at Winnipeg International Airport located approximately 35 miles (56 km) west. Figure 3 gives these maximum and minimum air temperature values at the two locations. The two sets of temperatures were used in COLD to see how close, with the same maximum and minimum values from the test site, prediction of temperatures compared with the detailed program could be. It was also desired to see whether program COLD could adequately simulate field conditions using available monthly meteorological records.

Two types of pavement structures, described by Christison (12), were considered. Structure B had a 4 inch (100 mm) asphaltic concrete surface, 16 inch (400 mm) granular base and clay subgrade and Structure D, a 10 inch

(250 mm) full depth asphaltic concrete and clay subgrade. The asphalt cement binders were of high and low viscosity types of the 150-200 penetration grade.

During the period of seasonal minimum ambient temperature, the predicted temperatures using the detailed program were roughly 4°F (2°C) higher than the recorded temperatures for Structure B, while the difference was ± 2°F (± 1°C) for Structure D. Using daily maximum and minimum temperatures at the test site, the COLD program predicted pavement temperatures in both structures that followed quite closely to the pattern of the recorded temperatures with only a few degrees variation. For a few extremely low temperatures, program COLD failed to predict the occurrence at the correct time.

In an effort to assess the thermal stress predictive capability of program COLD, comparisons were made between the cracking occurrence predicted by program COLD and the actual observed cracking at the same period of time.

The predicted time and magnitude of the thermally induced stresses for structure B with temperatures from Winnipeg International Airport and using low viscosity (LV) 150-200 and high viscosity (HV) 150-200 penetration grade asphalt cement are shown in Figures 4 and 5. Visual surveys indicated that initial cracking occurred on Dec. 31, 1967 and the majority of cracking from Jan. 1-7 of 1968 for LV 150-200 asphalt cement while no fracture occurred for the HV 150-200 section.

The predicted time and magnitude of the thermally induced stresses for structure D with temperatures from Winnipeg International Airport for the same two asphalt cements are shown in Figures 6 and 7. Crack sensors on site indicated initial cracking on Dec. 30, 1967 and subsequent cracking on Dec. 31 and Jan. 4, 1968 for the LV 150-200 penetration grade asphalt.

Thermal stresses predicted by program COLD using temperatures from the test site were greater in magnitude than those using the Winnipeg International Airport temperatures, nevertheless they give a good indication of when to expect cracking.

From these comparisons it may be concluded that the COLD program can predict pavement temperatures and thermal stresses very close to those predicted by the detailed program, and may be used as a distress prediction model with reasonable confidence.

Alberta Test Project - Fort Saskatchewan Highway 15-A-1. This project is a full-depth asphalt concrete pavement of 8 inches (200 mm) placed on a clay subgrade and constructed in the fall of 1971. The asphalt cement was specified as an AC-275, having a minimum viscosity of 275 poises at 140°F (60°C) and a penetration of 250 at 77°F (25°C). Cores taken about six years after construction indicated a significant variability in air voids. Average values from these were:

(1) Mile 8.25 to 13.37 - a penetration of 157 with 5.7 percent air voids and 16.8 percent voids in the mineral aggregate (VMA).

(2) Mile 1.14 to 6.26 - a penetration of 148 with 2.5 percent air voids and 14.6 percent voids in the mineral aggregate (VMA).

Tensile splitting tests were conducted on additional cores taken in 1978 at temperatures of 0°F (-18°C), -15°F (-26°C) and -30°F (-34°C) for the purpose of providing input data to the COLD program. No strength values are available for earlier pavement ages.

The COLD program was run for critical temperature periods of the first three winters, 1971-72, 1972-73 and 1973-74. Figure 8 shows the maximum and minimum air temperatures recorded at the Namao meteorological station only a few miles away, for the first winter. The minimum air temperature was -44°F (-42°C). The following two winters were relatively mild, having minimum air temperatures of only -25°F (-32°C) and -31°F (-35°C). Stiffness modulus values calculated from Van de Poels' nomograph at 7200 seconds loading time are shown in Table 1. Stress predictions for the first winter are shown in Figures 9 and 10. For the second section having 2.5 percent air voids, the COLD program predicts cracking for the first winter, but not for the other two. For the section with 5.7 percent air voids, no cracking was predicted for all three winters.

Visual surveys showed that minor cracking occurred the first winter and that additional cracking was evident in the following winters.

It can be concluded that the COLD program is only moderately successful in predicting cracking for this section of highway evaluated, using the input parameters as previously described. The fact that the sections had such differing air voids indicates a relatively high degree of variability and therefore the prediction of cracking could be based on other than 50 percent levels of reliability. The model has this capability (19), however this was not done in this study. Another possibility could be that the cracking which occurred in later years was caused by factors other than thermal shrinkage of the surface. It is known that the riding quality deteriorated quite rapidly due to swelling of the clay subgrade in the vicinity of the transverse cracks.

Test Sections in West Texas. Previous reports of extensive transverse cracking problems (22) have attributed thermal activity of the base course materials as being an important mechanism in the pavement cracking observed in West Texas. An investigation concerned primarily with the role of the asphalt concrete and the contributions to cracking made by the components, namely, the asphalt cement binder and the aggregate, has recently been completed (18). This provided an opportunity to compare predictions based on the COLD program with visual observations of pavement conditions after several years of service.

Two of the six test sections in the Odessa district of West Texas were used to provide input data to the COLD program. The stiffness modulus of the asphalt concrete was estimated for appropriate temperatures by using the nomographs of Van der Poel and Heukelom and Klomp as presented by McLeod (23). Again the loading time of 7200 seconds was used. Table 2 gives values for the two sections. The section on I-20 was selected as one exhibiting cracking with the section on US 285(186) as having no cracking. Properties of the asphalt concrete and recovered asphalt cement were determined on cores taken from test slabs removed at an age of 4 to 5 years. Direct tensile strength values were obtained at a loading rate of 0.02 in./min. (0.51 mm/min.) at temperatures of 75°F (24°C), 33°F (0.6°C) and -9°F (-23°C).

A partial summary of COLD program outputs for critical low temperature periods is given in Table 3. Examination of this table indicates that the thermally induced stresses exceed the expected strength of section I-20, but are far less than the expected strength for section US 285(186). This is compatible with the observed field performance of these two pavements. It can be concluded from these observations that the COLD program does produce predictions of low temperature cracking that are verified by two sections of pavements in West Texas.

Other Studies. A preliminary attempt at verification of the COLD program on pavements subjected to climatic conditions other than in Western Canada was made using available data presented on pavements in Pennsylvania by Kandhal (24). It was found that pavement temperatures could be simulated to a reasonable degree. Thermal fracture predictions could be made that conformed to visual evaluations reported when stiffness modulus values determined at a loading time of 200 seconds were used. This may be justified in part due to the rapid drops in temperature experienced on the Pennsylvania pavements.

Another instance of using the COLD program has been reported by Hode Keyser and Beraud on the James Bay project in Northern Quebec (25). This program was described as the Anderson-Christison method and apparently was not able to predict the extent and amount of low temperature cracking experienced. A review of the data used in this analysis has not yet been made at this time.

Summary

Extensive surveys of the main highway system in Alberta have indicated that larger amounts of transverse cracking are associated with certain asphalt sources. Restricting the use of "low viscosity" asphalt cements at a given penetration has resulted in a more uniform product being supplied and a reduction in the amount of transverse cracking.

Distress prediction methods based on induced stresses due to thermal shrinkage of the surface can be useful. The COLD program has been verified to some degree by field observations in Alberta and Manitoba as well as in Texas.

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Table 1

Alberta Test Project
Highway 15-A-1

Summary of Recovered Asphalt Cement Properties and Stiffness Modulus of Mix Estimated by Nomograph

Characteristic	Highway Section	
	Mi. 8.25 to 13.37	Mi. 1.14 to 6.26
Penetration @ 77°F (25°C), dmm	157	148
Softening Point, R&B °F (°C)	106 (41.2)	106 (41.4)
Penetration Index	-1.0	-1.0
Air Voids - percent	5.7	2.5
VMA - percent	16.8	14.6
Stiffness Modulus psi @ 7200 sec.		
Temperature 30°F (-1°C)	4,000	8,000
20°F (-7°C)	15,000	20,000
10°F (-12°C)	30,000	68,000
0°F (-18°C)	90,000	150,000
-10°F (-23°C)	250,000	560,000
-20°F (-29°C)	600,000	950,000
-30°F (-34°C)	1,400,000	2,000,000
-40°F (-40°C)	2,400,000	3,400,000

Table 2

West Texas Test Sections

Summary of Recovered Asphalt Cement Properties and Stiffness Modulus of Mix Estimated by Nomograph

Characteristic		Highway Section	
		I-20	US 285(186)
Penetration @ 77°F, (25°C), dmm		18	25
Softening Point, R&B °F (°C)		158 (70)	140 (60)
Penetration Index		-1.0	-1.0
Asphalt Content percentage by wt. of aggregate		7.3	7.7
Air Voids - percent		4.3	2.6
Stiffness Modulus psi @ 7200 sec.			
Temperature	75°F (24°C)	3,000	300
	50°F (10°C)	70,000	7,000
	33°F (0.6°C)	500,000	48,000
	10°F (-12.2°C)	1,800,000	500,000
	-9°F (-23°C)	2,000,000	1,800,000

Table 3

West Texas Test Sections

Summary of COLD Program Outputs
for Critical Low Temperature Periods

Date		January 5, 1971	
Day		12	
Time Period	133	134	135
Air Temperature, °F	2	6.7	11.3
Section I-20			
Pavement Temperature, °F	17.2	17.0	18.6
Tensile Strength, psi	275.8	276.1	274.2
Induced Stress, psi	343.5*	349.4*	311.5*
Section US 285(186)			
Pavement Temperature, °F	18.4	17.8	19.5
Tensile Strength, psi	319.2	322.5	313.8
Induced Stress, psi	34.1	36.1	30.6

* Indicates that expected thermally induced stress has exceeded expected strength.

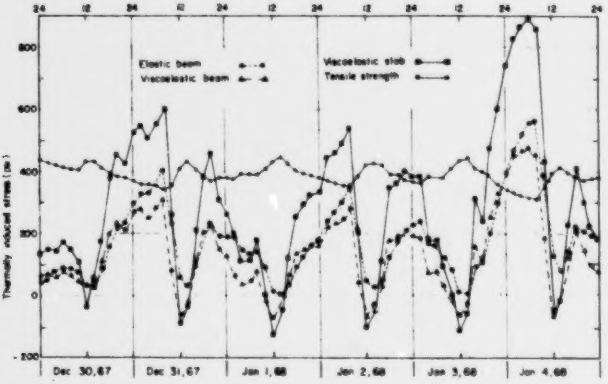

Fig. 1. Thermally Induced Stress at 1/2 Inch Depth of the LV 150-200 Asphaltic Concrete Pavement Surface of Structure D (after Christison et al., Ref. 14)

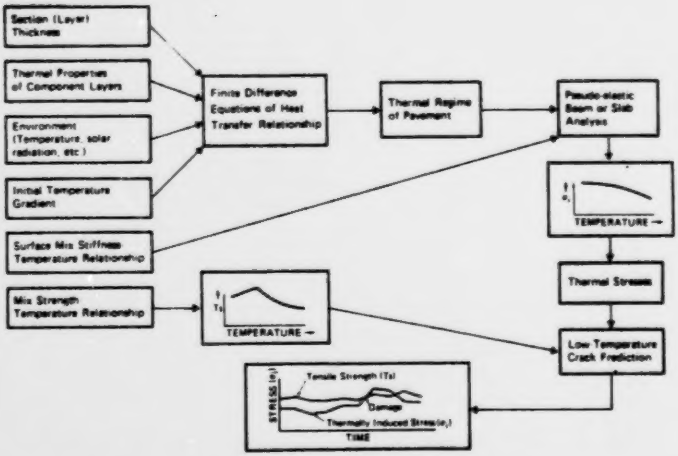
Section (Layer) Thickness
Thermal Properties of Component Layers
Environment (Temperature, solar radiation, etc.)
Initial Temperature Gradient
Surface Mix Stiffness Temperature Relationship
Mix Strength Temperature Relationship
Finite Difference Equations of Heat Transfer Relationship
Thermal Regime of Pavement
Pseudo-elastic Beam or Slab Analysis
TEMPERATURE →
Thermal Stresses
Low Temperature Crack Prediction
Ts
TEMPERATURE →
STRESS (σt)
Tensile Strength (Ts)
Damage
Thermally Induced Stress (σt)
TIME

Fig. 2. Low-Temperature Cracking (after Finn et al., Ref. 16)

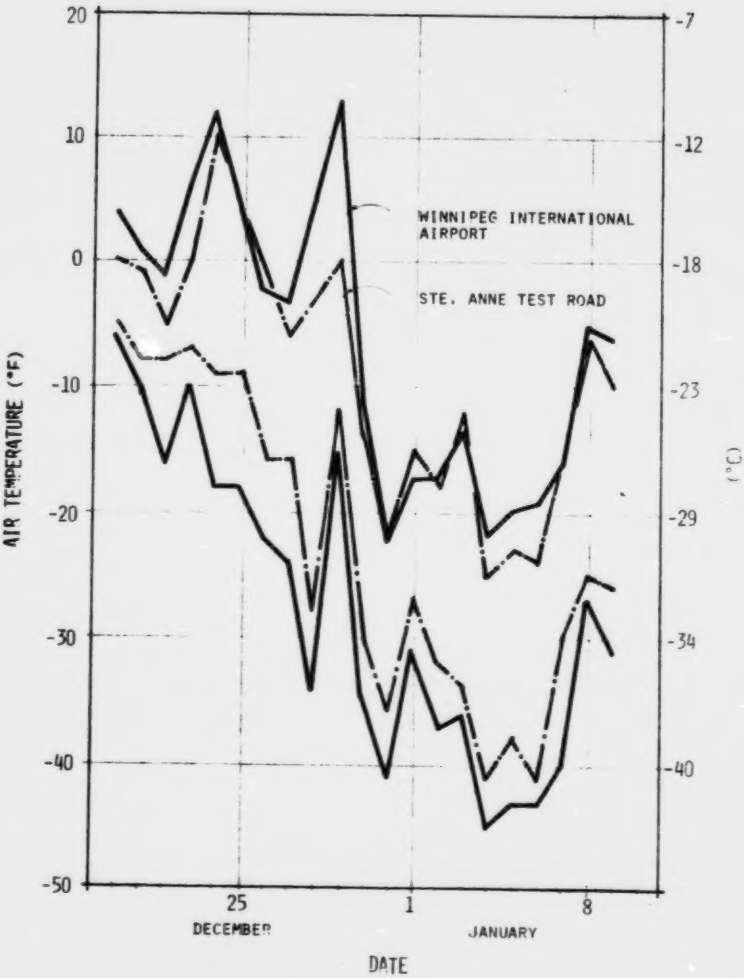

Fig. 3. Maximum and Minimum Temperatures Recorded at Ste. Anne Test Road and Winnipeg International Airport

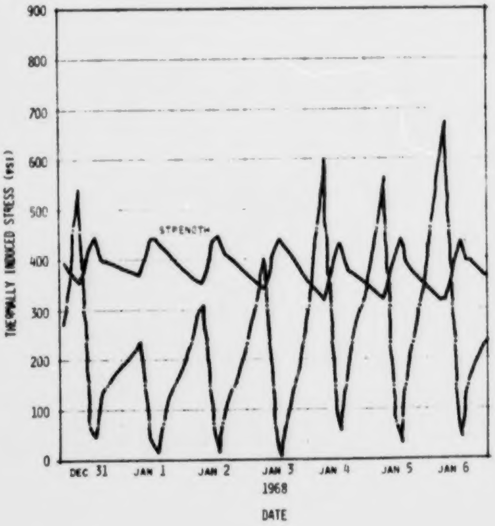

Fig. 4. Thermally Induced Stresses at 1/2 Inch Depth of the LV 150-200 Pavement Surface of Structure B

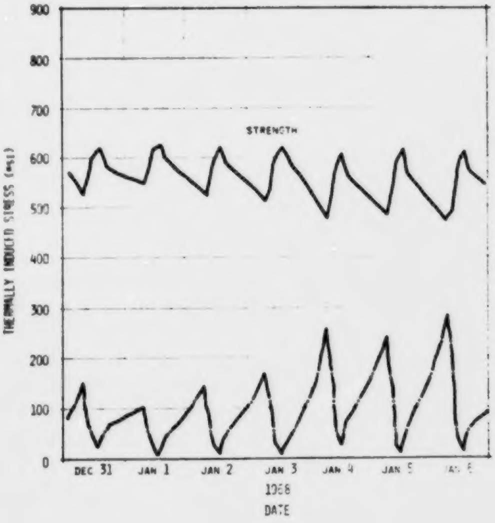

Fig. 5. Thermally Induced Stresses at 1/2 Inch Depth of the HV 150-200 Pavement Surface of Structure B

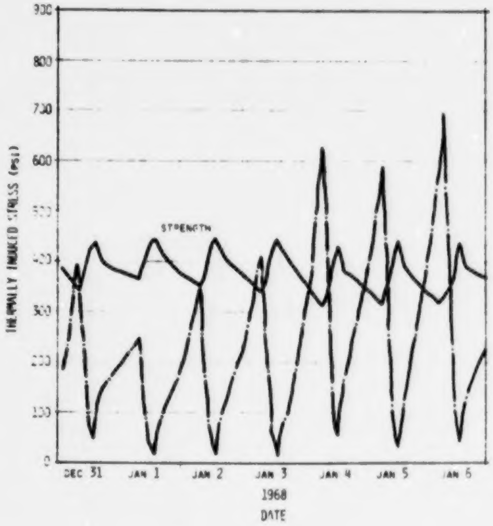

Fig. 6. Thermally Induced Stress at 1/2 Inch Depth of the LV 150-200 Pavement Surface of Structure D

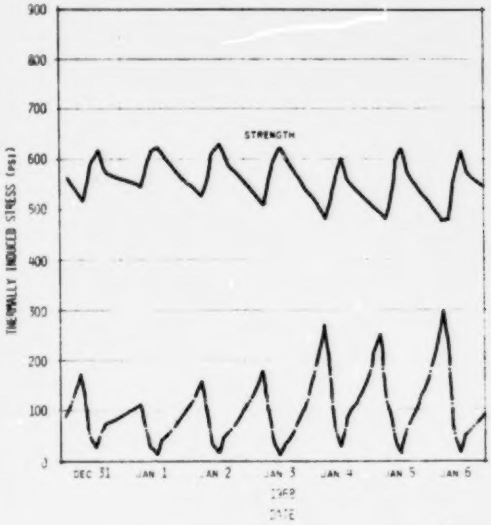

Fig. 7. Thermally Induced Stress at 1/2 Inch Depth of the HV 150-200 Pavement Surface of Structure D

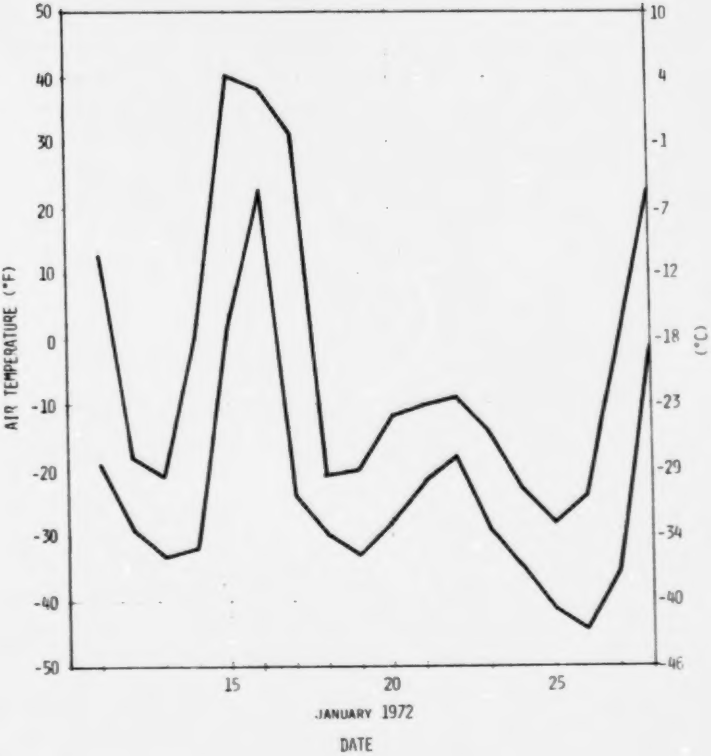

Fig. 8. Maximum and Minimum Air Temperatures Recorded at Namao Meteorological Station

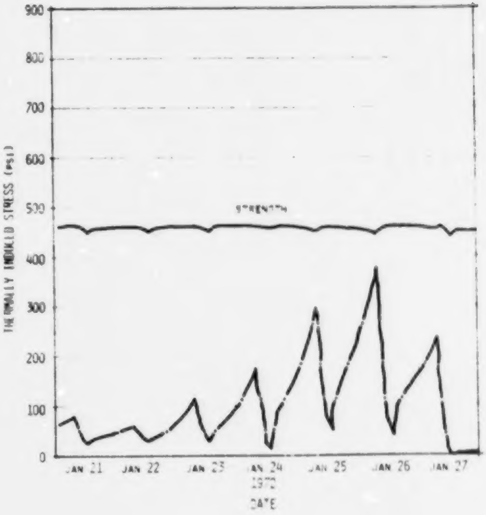

Fig. 9. Thermally Induced Stresses at the Pavement Surface of Highway 15-A-1, Mile 8.25 to 13.37

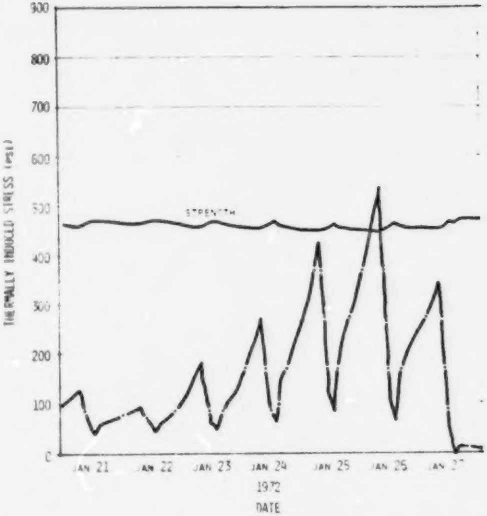

Fig. 10. Thermally Induced Stresses at the Pavement Surface of Highway 15-A-1, Mile 1.14 to 6.26

Session 1

Thermally Induced Cracking Due to Hardening of Asphalt Concretes in Airport Asphalt Pavements

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ABSTRACT

Low temperature cracking of asphalt pavements is a serious problem in airports throughout the northern part of Japan. The actual conditions of low temperature cracking in two airports have been investigated. Low temperature cracking is more severe in non-traffic areas such as the edge part of runways and taxiways than in traffic areas. Generally, the aging or hardening of asphalt concrete is faster in non-traffic areas than in traffic areas. This report deals with the effect of the aging of asphalt concrete on low temperature cracking.

Some experiments have been conducted to evaluate the effects of aging on the behavior of asphalt concrete under thermally induced stresses. In fresh asphalt concrete with no aging, there has been no increase in thermally induced stresses with thermal cycling (-20°C-10°C, 100 cycles) and no influence on the mechanical properties such as strength and strain at failure. On the other hand, in asphalt concrete with aging, there have been influences on the mechanical properties and the thermally induced stresses.

From these kinds of results, it has been presumed that the aging or hardening of asphalt concrete may have a large effect on low temperature cracking in airport asphalt pavements and the prevention of aging or hardening of asphalt concrete may be a good way to decrease the extent of low temperature cracking.

THERMALLY INDUCED CRACKING DUE TO HARDENING OF ASPHALT CONCRETES IN AIRPORT ASPHALT PAVEMENTS

by

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1. Introduction

It is characteristic of airport pavement that traffic concentrates considerably in limited areas such as the center portion of a runway or taxiway while the remaining large area seldom has traffic. It is believed that the aging or hardening of the asphalt concrete of the surface course in the non-traffic area progresses faster than that in the traffic area since the former scarcely receives any kneading action from traffic.

Thermally induced cracking, closely related to the hardening of asphalt concrete, has occurred in some airports in Japan. In the airports in cold areas, low temperature cracking has sometimes occurred, because the thermally induced stresses are large due to the severe climatic conditions.

This report shows how the hardening of asphalt concrete influences the cracking of asphalt concrete due to thermally induced stresses. A serie of laboratory tests were conducted on various types of asphalt concretes composed of combinations of two types of asphalt, two different asphalt contents and two rates of aggregate absorption to evaluate the effects of hardening of asphalt concrete due to heat curing related to thermally induced stresses. The test results made clear, to a certain degree, the effects of hardening of asphalt concrete on thermally induced cracking.

2. Actual condition of hardening of asphalt concrete and thermally induced cracking in airport asphalt pavements

Fig.2.1 is the location of the airports which were investigated in this study.

Fig.2.2 shows the typical cracking condition of asphalt pavement at the runway edge and shoulder at Kushiro Airport, which is located in Hokkaido and which has quite a severe winter (freezing index, 591.8°C·day). The age of this asphalt pavement is about 10 years. The cracks in the runway edge

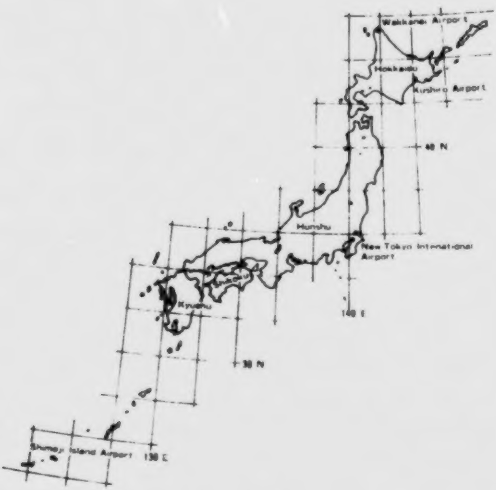
Wakkanai Airport
Hokkaido
Kushiro Airport
40 N
Honshu
New Tokyo International Airport
Shikoku
Kyushu
140 E
30 N
Shimoji Island Airport
130 E

Fig.2.1 Locations of airports for investigations

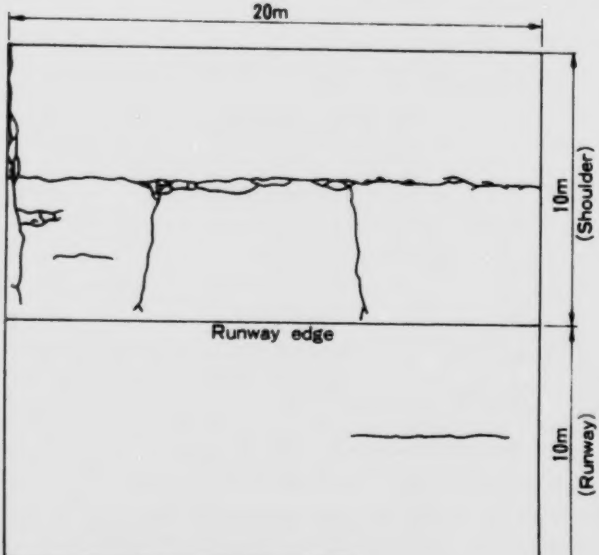
20m
10m
(Shoulder)
Runway edge
10m
(Runway)

Fig.2.2 Example of cracking at Kushiro Airport

are few, but the shoulder has many. These cracks are not only thermally induced but are also due to frost heave. All in all, it is thought that thermally induced cracks are not so remarkable at this airport. Besides, at Wakkanai Airport, which is colder than Kushiro Airport (freezing index, 657.6°C·day), two year old asphalt pavement seldom has thermally induced cracks.

As can be seen from the above results, the airport asphalt pavements in cold areas in Japan have a few thermally induced cracks, but the situation is not too severe. It is assumed that the reason for this may be closely related to the composition of asphalt concrete used in airports in Japan. The aggregate gradations are prescribed as shown in Table 2.1. These are rather fine. Also, Table 2.2 shows the specifications of the Marshall test for asphalt concrete, and the asphalt content is determined and made as high as possible in consideration of the age-hardening of asphalt concrete. Table 2.3 shows the physical properties of the asphalt extracted from mixes at Kushiro Airport. It is recognized from these results·that the degree of the age-hardening of the asphalt concrete is average.

Table 2.1 Aggregate gradation for surface course

Sieve Designation (mm)	Percentage by Weight Passing Sieves	
	A	B
	20 mm Maximum	13 mm Maximum
25	100	
20	95∿100	100
13	75∿ 90	95∿100
5	45∿ 65	55∿ 75
2.5	35∿ 50	35∿ 50
0.6	18∿ 29	18∿ 29
0.3	10∿ 21	10∿ 21
0.15	6∿ 16	6∿ 16
0.074	4∿ 8	4∿ 8

Table 2.2 Requirements of Marshall test for surface course

Number of blows	75
Stability (kgf) (Minimum)	900
Flow (1/100 cm)	20 ∿ 40
Percent voids	2 ∿ 5
Percent voids filled with asphalt	75 ∿ 85

Table 2.3 Physical properties of recovered asphalt at Kushiro Airport

Section (cracking condition)	Sample	Penetration (1/10 mm, 25°C)	Softening Point (°C)	Penetration Index	Ductility (cm, 15°C)
	Origin	93	46	-0.7	100+
Low	0 ∿ 10 mm	31	58.0	-0.4	12
	13 ∿ 23 mm	30	57.0	-0.7	15
	26 ∿ 36 mm	42	54.5	-0.5	22
Medium	0 ∿ 10 mm	26	59.0	-0.6	10
	13 ∿ 23 mm	34	56.5	-0.6	13
	26 ∿ 36 mm	44	55.0	-0.3	18
High	0 ∿ 10 mm	29	61.5	0.1	5
	13 ∿ 23 mm	30	57.0	-0.7	5
	26 ∿ 36 mm	41	55.5	-0.4	15

On the other hand, thermally induced cracks extraordinarily occurred in the asphalt pavements of runways or taxiways in some airports located in rather warm areas. These cracks are believed to be a kind of thermally induced crack since the cracking was before service and the pavement had no traffic.

Table 2.4 is the cracking degree of asphalt pavements at New Tokyo International Airport in 1975, three to five years after construction. Table 2.5 shows the physical properties of the asphalts extracted from the mixes. The hardening of the asphalt from the mix of A Runway, which has more cracking than other areas, is larger than that in the other areas, especially the surface portion (0∿5 mm).

Table 2.4 Cracking condition at New Tokyo International Airport

Section	Cracking Ratio (%)				Asphalt Type
	Blistering	Longitudinal & Transverse	Alligator	Total	
A Runway	0.10	2.01	0.54	2.65	Semi-blown
C Runway	0.31	0.15	——	0.46	Straight
P Taxiway	——	0.25	——	0.25	Straight

$$\text{Cracking Ratio } (\%) = \frac{\text{Cracking Area } (m^2)}{\text{Total Pavement Area } (m^2)} \times 100$$

$$\text{Cracking Area of Blistering } (m^2) = 0.09 \times \text{Number of Blisterings}$$

$$\text{Cracking Area of Linear Cracks } (m^2) = 0.3 \times \text{Length of Cracks}$$

Table 2.5 Physical properties of recovered asphalt at New Tokyo International Airport

Asphalt Type	Section	Sample	Penetration (1/10 mm, 25°C)	Softening Point (°C)	Penetration Index	Ductility (cm, 15°C)
Semi-blown	A Runway	Origin	70	49.5	-0.5	——
		0∿ 5 mm	16	74.6	1.1	1.0
		8∿13 mm	31	61.8	0.3	4.5
		16∿35 mm	47	57.7	0.4	9.0
Straight	C Runway	Origin	65	52.0	-0.1	——
		0∿ 5 mm	19	56.7	-1.6	4.0
		8∿13 mm	41	50.7	-1.5	25.0
		16∿35 mm	55	48.3	-1.4	100+
	P Taxiway	Origin	65	52.0	-0.1	——
		0∿ 5 mm	16	61.3	-1.0	0.5
		8∿13 mm	25	56.5	-1.2	5.0
		16∿35 mm	51	50.5	-1.0	26.5

Fig.2.3 shows the cracking condition of the asphalt pavement of the taxiway at Shimoji Island Airport. The cracking began about nine months after the completion of the construction, and the cracks are presumed to be a kind of thermally induced crack since the pavement cracked before service and had no traffic at all. The reason for the cracking is thought to be the asphalt

concrete used for the surface course. Coral reef rock was used for aggregate in spite of its high absorption characteristic (coefficient of water absorption, 2.9%), since it is the only kind of rock found on that island, and semi-blown asphalt (penetration, 47: softening point, 69.5°C) was used for binder in consideration of rutting. The penetration and softening point of the asphalt extracted from the mixes after cracking are, respectively, 18 and 125°C. Therefore, the hardening of the asphalt concrete would be expected to be considerably great.

On the basis of the above-mentioned observations, thermally induced cracks of asphalt pavements might occur even in airports in comparatively warm areas, if the hardening of asphalt concrete has progressed to a high degree. From this point of view, the consideration of the hardening of asphalt concrete is very important, especially in the non-traffic area of airport pavements.

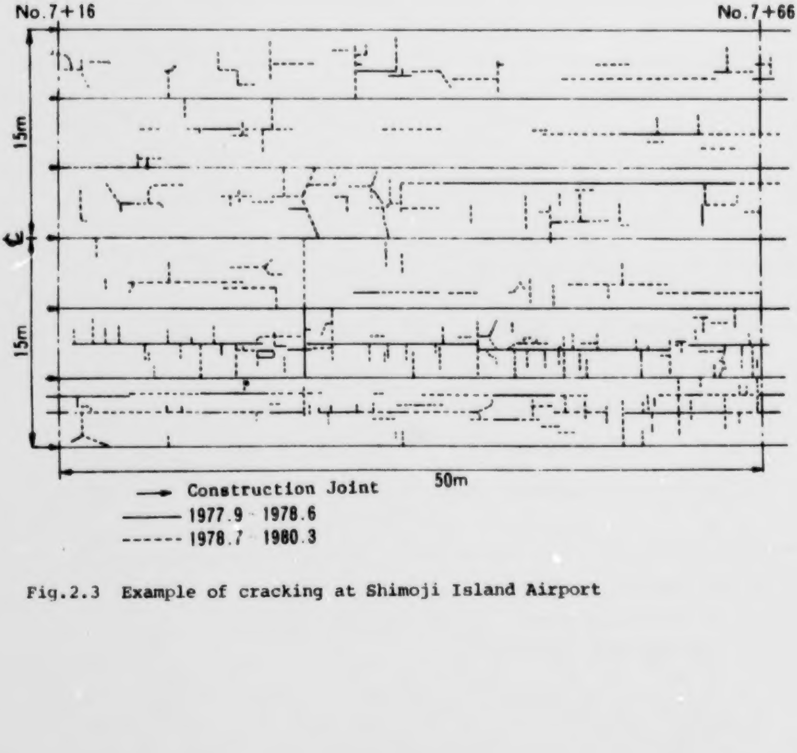

Fig.2.3 Example of cracking at Shimoji Island Airport

3. Laboratory test

In order to investigate the effects of heat and thermal cycling on the mechanical behavior of asphalt concrete and on the physical properties of asphalt, a series of laboratory tests were conducted. Materials used in the tests and test methods are described below.

3.1 Materials

Table 3.1 shows the types of asphalt concretes used in the laboratory tests. Two types of aggregates were used. One is normal aggregate which consists mainly of andesite, and the other is coral reef rock. Their coefficients of water absorption are 2.28% and 9.51% respectively. Their gradations are shown in Fig.3.1. Also, two types of asphalts, straight asphalt 60/80 and semi-blown asphalt 40/60 were used, and their physical properties are shown in Table 3.2.

3.2 Test methods

First of all, the effects of heat curing at 70°C were investigated. Flexural tests at a constant rate of strain and measurements of the coefficient of thermal expansion of asphalt concretes were conducted, and the physical properties of asphalts extracted from the asphalt concretes were examined.

Next, the stresses induced by thermal cycling (+10°C to -20°C, 100 cycles) in asphalt concretes which were both heat cured and non-heat cured were ob-

Table 3.1 Types of asphalt concretes used in laboratory tests

Asphalt Concrete	Asphalt	Aggregate	Asphalt Content (%)
NS	Straight 60/80	Normal	6.2 (OAC)
CS	Straight 60/80	Coral Reef Rock	9.3 (OAC)
CS + 1	Straight 60/80	Coral Reef Rock	10.3 (OAC + 1)
NP	Semi-Blown 40/60	Normal	6.2 (OAC)
CP	Semi-Blown 40/60	Coral Reef Rock	9.3 (OAC)

N: Normal Aggregate S: Straight Asphalt
C: Coral Reef Rock P: Semi-blown Asphalt
OAC: Optimum Asphalt Content

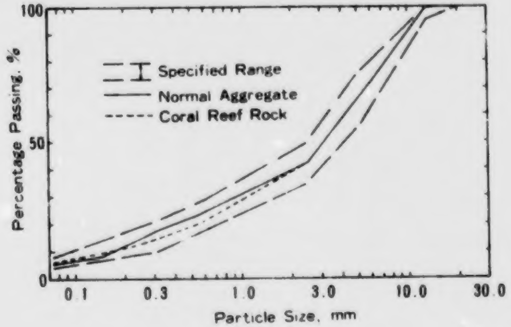

Fig.3.1 Gradations of aggregates used in laboratory tests

Table 3.2 Physical properties of asphalts used in laboratory tests

Physical Property		Straight 60/80	Semi-blown 40/60
Penetration (1/10 mm, 25°C)		68	50
Softening Point (°C)		49.5	59.0
Ductility (cm, 15°C)		140+	——
Penetration Index		-0.6	0.8
Viscosity (cst)	120°C	670	——
	140°C	280	704
	160°C	139	294
	180°C	77	131
Specific Gravity		1.029	1.021

served, and then flexural tests were carried out at a constant rate of strain on the asphalt concretes to investigate the fatigue phenomena due to thermal cycling.

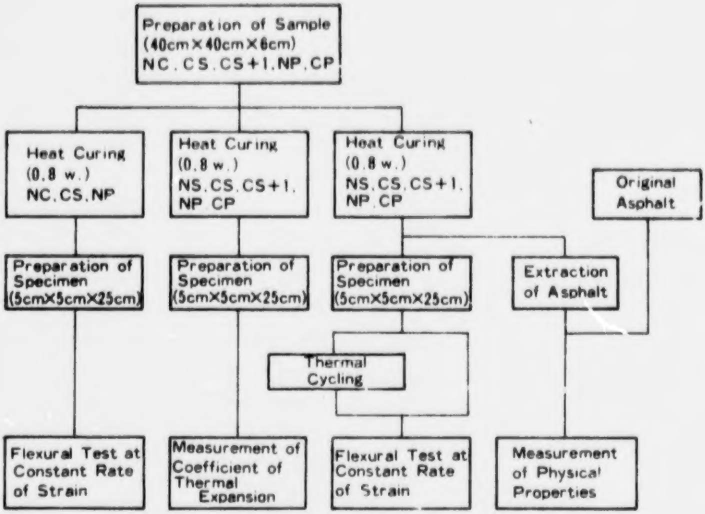
Preparation of Sample
(40cm×40cm×6cm)
NC.CS.CS+1.NP.CP
Heat Curing
(0.8 w.)
NC.CS.NP
Heat Curing
(0.8 w.)
NS.CS.CS+1.
NP.CP
Heat Curing
(0.8 w.)
NS.CS.CS+1.
NP.CP
Original
Asphalt
Preparation of
Specimen
(5cm×5cm×25cm)
Preparation of
Specimen
(5cm×5cm×25cm)
Preparation of
Specimen
(5cm×5cm×25cm)
Extraction
of Asphalt
Thermal
Cycling
Flexural Test at
Constant Rate
of Strain
Measurement of
Coefficient of
Thermal
Expansion
Flexural Test at
Constant Rate
of Strain
Measurement
of Physical
Properties

Fig.3.2 Flowchart of laboratory tests

Fig.3.2 shows a flowchart of these tests. Individual items in this flowchart are described below.

(1) Asphalt concrete specimens

Firstly, asphalt concrete samples whose types are shown in Table 3.1 were compacted into a 40 cm × 40 cm × 6 cm slab to above 98% of the density specified by the Marshall test. After the prescribed heat curing, the sample was cut into specimens 4 cm × 4 cm × 25 cm in size.

(2) Heat curing

Heat curing was carried out by curing the asphalt concrete samples in an oven at a constant temperature of 70°C for specified periods. These periods were eight weeks for specimens which are used in the flexural test at a constant rate of strain to investigate only the effects of heat curing and in the measurement of the coefficient of thermal expansion, and 1, 2, 4 and 8 weeks for specimens used in the other tests.

(3) Measurement of thermally induced stress

Five types of asphalt concretes shown in Table 3.1, which were both heat cured and non-heat cured, were subjected to thermal cycling. The procedure is as follows. Firstly, an asphalt concrete specimen was attached to a frame made of invar having a very low coefficient of thermal expansion to minimize the change of length of the specimen (Fig.3.3). Next, the specimen with frame was soaked in non-freezing liquid, and then subjected to thermal cycling of +10°C ∿ -20°C ∿ +10°C (+20°C/h at the temperature rising and -16°C/h at the temperature falling). By means of strain gauges attached to the frame, thermally induced stress was measured.

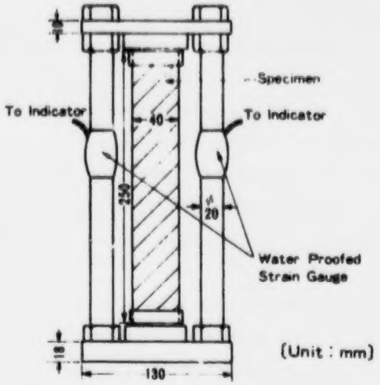

Fig.3.3 Frame used in measuring thermally induced stresses

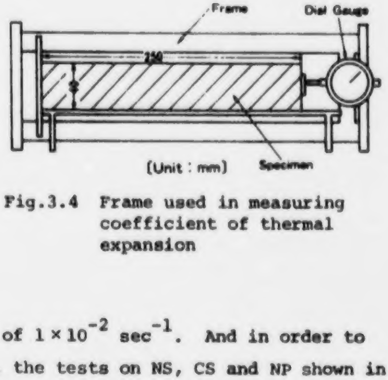

Fig.3.4 Frame used in measuring coefficient of thermal expansion

(4) Flexural test at a constant rate of strain

Flexural tests at a constant rate of strain on specimens subjected to thermal cycling were conducted at a temperature of 10°C with a strain rate of 1×10^{-2} sec^{-1}. And in order to investigate the effects of heat curing, the tests on NS, CS and NP shown in Table 3.1, some of which were heat cured and some non-heat cured, were carried out at temperature of -20, -10, 0, 10 and 20°C with strain rates of 5×10^{-5}, 1×10^{-4}, 1×10^{-3} and 1×10^{-2} sec^{-1}.

(5) Measurement of the coefficient of thermal expansion

The frame shown in Fig.3.4 was used in measuring the coefficient of thermal expansion of five types of asphalt concretes shown in Table 3.1, some heat cured and some non-heat cured. The procedure consists of placing a specimen in a temperature controlled cabinet, where the temperature was changed cyclically from +10°C to -20°C and back, and then measuring the length of the specimen using the measuring frame at specified temperatures.

(6) Physical property tests on asphalts

Asphalts were extracted using the Abson method specified in ASTM D 1856 from the five types of asphalt concretes shown in Table 3.1, some being heat cured and some non-heat cured. And then physical property tests were conducted on them. These tests are listed below.

(a) Viscosity test: in accordance with ASTM D 2171
(b) Penetration test, Softening point test and Fraas breaking point test: in accordance with JIS K 2207
(c) Compositional analysis: asphalt is fractionated to asphaltene, resin, aromatic and saturate using the Chromatograph method.
(d) Infrared ray analysis: a solution of asphaltene in CCl_4 is placed between disks made of NaCl, and then an infrared ray is applied to it.

4. Results and discussion

4.1 Change of properties of asphalt concrete and asphalt bitumen with curing time

(1) Change in mechanical properties of asphalt concrete

From a series of flexural tests at a constant rate of strain, flexural strength, strain at failure and stiffness at 0.1% of strain were obtained. The rate of strain was 1×10^{-3} sec^{-1} and the temperature was 10°C for these tests.

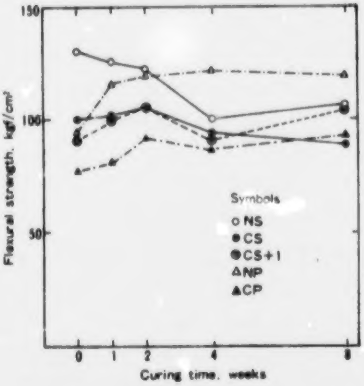

Fig.4.1 Influence of curing time on flexural strength

Fig.4.1 shows the relationship between the flexural strength and curing time. The results show that asphalt concretes using semi-blown asphalt increase their strength with

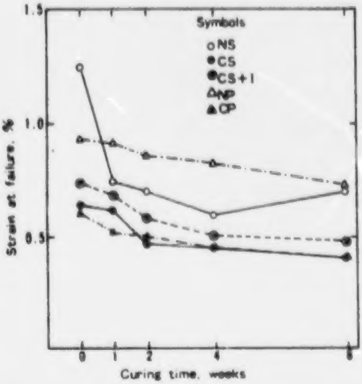

Fig.4.2 Influence of curing time on strain at failure

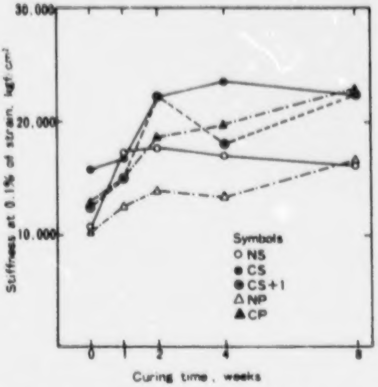

Fig.4.3 Influence of curing time on stiffness at 0.1% of strain

curing time. On the other hand, asphalt concretes using straight asphalt have nearly constant or gradually decreasing strength with curing time. For both straight and semi-blown asphalt, the change of strength is larger when normal aggregate is used than when coral reef rock is used.

Fig.4.2 shows the change of the strain at failure, and it is clear that it decreases with curing time. Except for the first week of NS, the slopes of the curves are almost the same.

The change of the stiffnesses are shown in Fig.4.3. Every mixture has increased stiffness with curing time. Coral reef rock has especially significant effects on these phenomena, and stiffness increases up to 22,000 kgf/cm^2 during the eight weeks of curing regardless of the type or the amount of asphalt bitumen. The reason might be that coral reef rock has a large capacity for asphalt absorption and tends to harden the asphalt bitumen more compared with normal aggregate.

Judging from Fig.4.1 to Fig.4.3, asphalt concrete seems to get hard and brittle owing to curing at the temperature of 70°C in all cases.

Another series of flexural tests was performed on NS, CS and NP under a wide range of rates of strain and temperatures. The relationship between

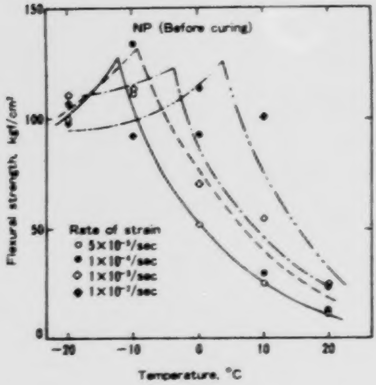

Fig.4.4(a) Effects of temperature on flexural strength (Before curing)

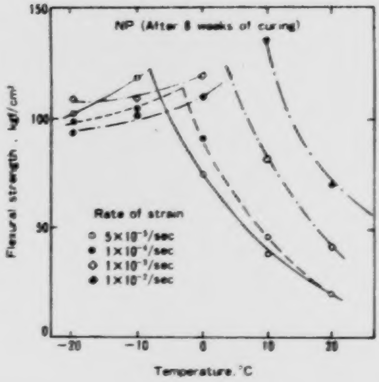

Fig.4.4(b) Effects of temperature on flexural strength (After 8 weeks of curing)

flexural strength and temperature for NP is shown in Fig.4.4(a) and Fig.4.4(b). These figures show that the transition point, which is defined as the temperature corresponding to the peak of the strength-temperature curve, moves toward the right. This indicates that asphalt concretes become hard during the eight weeks of curing.

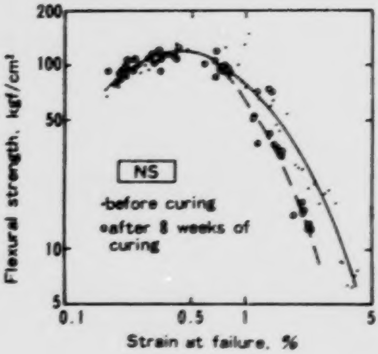

Fig.4.5(a) Failure envelope (NS)

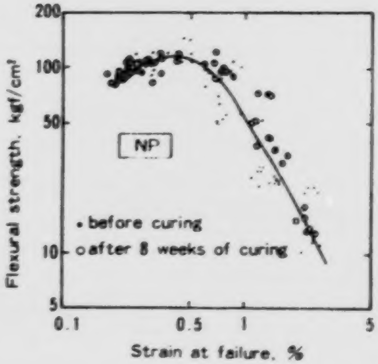

Fig.4.5(b) Failure envelope (NP)

In Fig.4.5, the relationship between flexural strength and strain at failure is plotted. This kind of diagram is called a failure envelope and is said to be a unique curve for any given mixtures of asphalt concrete regardless of the rate of strain, temperature or the type of failure. Fig.4.5 confirms this. However, for the asphalt concrete using straight asphalt, the failure envelope moves slightly toward the left over the range of strain higher than 0.5%. This means that the allowable strain for a given level of stress decreases, and asphalt concrete becomes brittle owing to eight weeks of curing. On the other hand, for asphalt concrete using semi-blown asphalt, curing for eight weeks seems to have little effect on the location or the shape of its failure envelope.

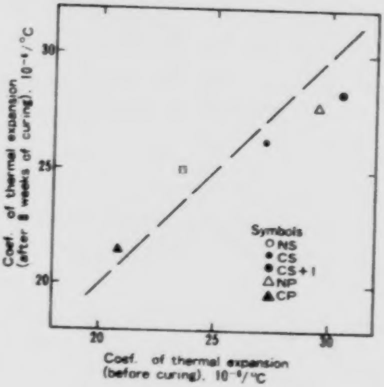

Fig.4.6 Comparison of coefficients of thermal expansion

(2) Coefficient of thermal expansion of asphalt concrete

Fig.4.6 shows the comparison of the coefficients of thermal expansion of each asphalt concrete measured before and after eight weeks of curing. The effects on the coefficient of curing, the type of aggregate and binder are not clear from Fig.4.6. The coefficient of thermal expansion of CS + 1 is larger than that of CS in Fig. 4.6, and this may be caused by the high asphalt content of CS + 1, because the coefficient of thermal expansion of asphalt bitumen is usualy larger than that of aggregate.

(3) Change in physical properties and chemical composition of asphalt bitumen

Fig.4.7 to Fig.4.10 show the effects of curing time on the physical properties of recovered asphalt, such as residual penetration, viscosity at 60°C, penetration index PI and Fraas breaking point. Each property indicates that asphalt becomes hard and decreases its temperature susceptibility

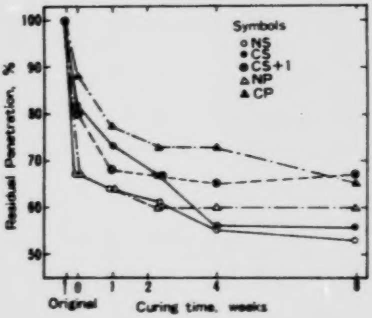

Fig.4.7 Influence of curing time on residual penetration

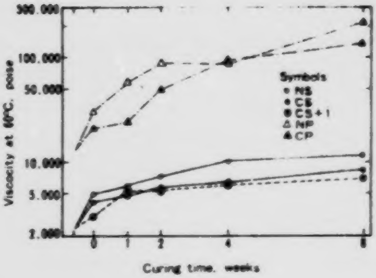

Fig.4.8 Influence of curing time on viscosity at 60°C

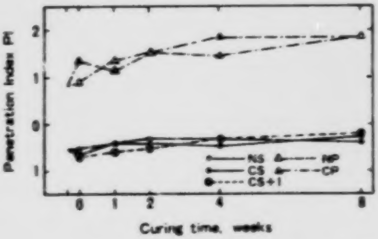

Fig.4.9 Influence of curing time on penetration index PI

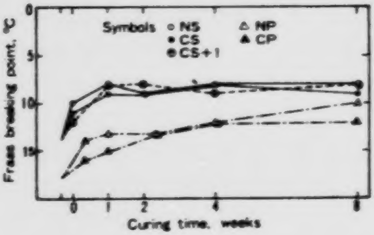

Fig.4.10 Influence of curing time on Fraas breaking point

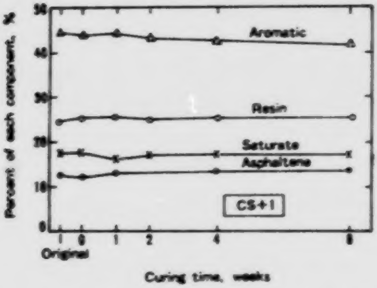

Fig.4.11(a) Influence of curing time on chemical composition (CS + 1)

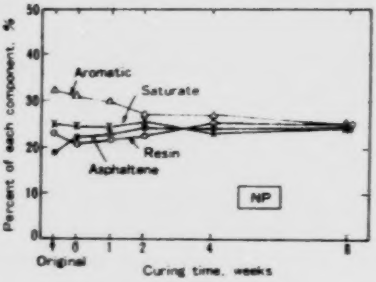

Fig.4.11(b) Influence of curing time on chemical composition (NP)

with curing time. Fig.4.7 also shows that asphalt bitumen mixed with normal aggregate drops its penetration to 70% of the original value at initial mixing of materials, regardless of the type of asphalt. This seems to indicate that less asphalt content in the asphalt mixture makes thinner asphalt film around the aggregate particles and accelerates the rate of hardening of asphalt during the mixing process.

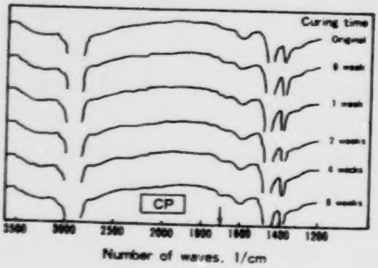

Fig.4.12 Results of infrared ray analyses (CP)

Some results of compositional analyses are shown in Fig.4.11(a) and Fig.4.11 (b). Basically, these figures reveal the significant differences in chemical composition between straight and semi-blown asphalts. For both types of asphalt, the compositional changes due to curing decrease in aromatic and increase in asphaltene. A comparison of these results to other mixtures shows that the proportion of each component seems to coincide for the same type of asphalt and curing time, regardless of the aggregate type or the asphalt content.

Some results of infrared ray analyses performed to examine the degree of oxidation of asphalt are shown in Fig.4.12. For the pavements in service, oxidation of asphalt is also one of the major factors for hardening and/or aging of asphalt bitumen other than heat effects. It is reported that a peak based on the carbonyl group ($C=O$) appears at 1700 1/cm for highly oxidized asphalt. In Fig.4.12, however, a clear peak cannot be recognized and oxidation of asphalt does not seem to progress during the eight weeks of curing in 70°C.

Judging from these results, there seems to be no significant effect on oxidation of asphalt in the employed test procedure, although it accelerates the change in physical properties and chemical composition. For further research an accelerated test procedure to simulate aging effects of asphalt concrete caused by both heat effects and oxidation of asphalt bitumen is expected to be developed.

4.2 Factors affecting thermally induced stresses in asphalt concrete

A series of thermally induced stresses up to 100 cycles was loaded on five kinds of mixtures in accordance with the procedure explained in 3.2(3). Stresses during the first cycle of temperature change will now be discussed.

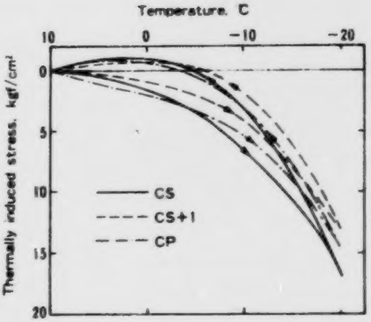

Fig.4.13 Measured thermally induced stresses

In Fig.4.13, the changes of thermally induced stresses with temperature are illustrated for the specimens without curing. Every curve shows a counter-clockwise hysteresis loop. In Fig.4.13, it appears consistent that a larger stress is induced in the mixture using straight asphalt than in that using semi-blown asphalt at -20°C, although the former has higher penetration and lower viscosity at 60°C than the latter. This phenomenon might be owing to the difference of the temperature susceptibility between these two asphalts. Fig.4.13 also shows that a lower stress is induced in the mixture with higher asphalt content having a larger coefficient of thermal expansion. This seems to be owing to the lower value of relaxation modulus for the mixture with higher asphalt content.

The influence of curing time on thermally induced stresses at -20°C is shown in Fig.4.14. Although these data are scattered, stresses generally increase with curing time because of hardening effects of curing.

Fig.4.15 shows the relationship between thermally induced stresses and softening point of recovered asphalt. For both types of asphalt, there seems to be a relatively close correlation between stress and softening point. It might be possible to compare the intensity of thermally induced stresses if the type of asphalt and softening point are given. On the other hand, no correlations as strong as those shown in Fig.4.15 can be obtained with other physical properties, such as penetration, Fraas breaking point or

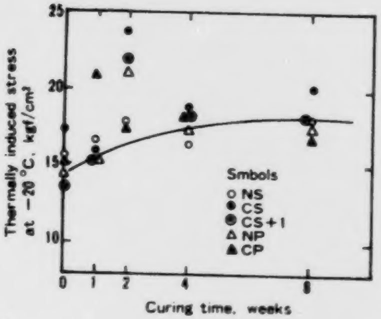

Fig.4.14 Influence of curing time on thermally induced stresses at -20°C

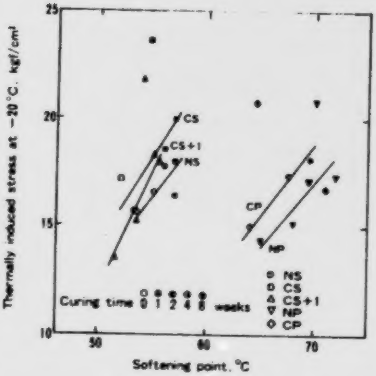

Fig.4.15 Relationship of thermally induced stress and softening point of asphalt

viscosity at 60°C.

4.3 Simulation of thermally induced stresses

In order to predict thermally induced stresses under different temperature conditions, simulation was performed by computer using the linear viscoelastic theory. For the calculation of thermally induced stress, the relaxation modulus $E_r(t,T)$ as a function of time t and temperature T, and the coefficient of thermal expansion α are necessary. Relaxation moduli were obtained from a series of relaxation tests and the time-temperature superposition principle as shown in Fig.4.16. For the coefficient of thermal expansion, 25×10^{-6}/°C was employed for both asphalt concretes.

The following procedure was employed for the calculation of thermally induced stresses. Strain increment $\Delta\varepsilon$ due to a temperature interval ΔT can be expressed in $\alpha\Delta T$ for an unrestrained beam. If the contraction and expansion of the beam are restricted, a stress $\Delta\sigma(t)$ is induced in the beam and it is expressed as follows,

$$\Delta\sigma(t)=\Delta\varepsilon E_r(t,T)=\alpha\Delta T E_r(t,T)$$

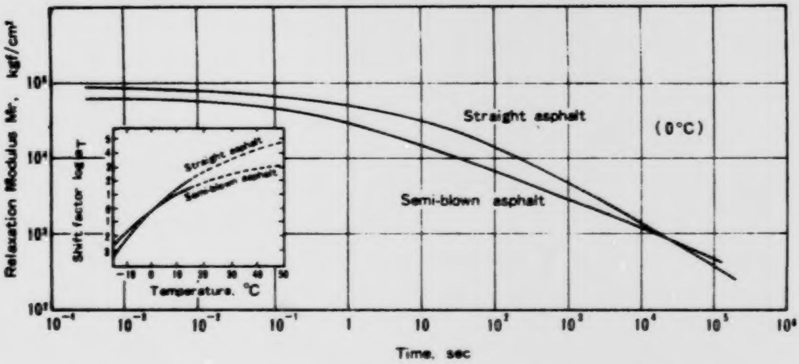

Fig.4.16 Master curves of relaxation modulus used for simulation

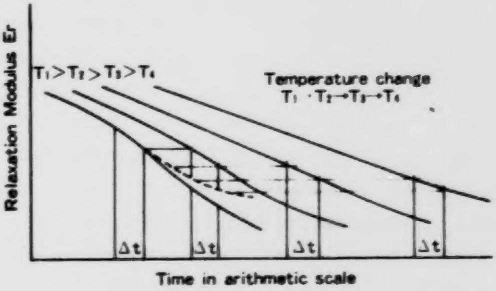
Relaxation Modulus Er
T₁>T₂>T₃>T₄
Temperature change
T₁→T₂→T₃→T₄
Δt
Δt
Δt
Δt
Time in arithmetic scale

Fig.4.17 Determination of relaxation modulus with temperature variation

Relaxation moduli resulting from temperature variation with time were obtained by shifting their master curves along the axis of time in accordance with the sequence of temperature variations as shown in Fig. 4.17. By superimposing this stress sequence, thermally induced stresses as a function of time can be calculated.

Fig.4.18 shows the results simulated by this procedure for the condition given in 3.2(3). The effects of elastic deformation (3.75 kgf/10^{-6}) and the coefficient of thermal expansion (1 ×10^{-6}/°C) of the invar frame were considered in the calculation process. It is revealed in Fig.4.18 that the temperature-stress curve shows a counter-clockwise hysteresis loop and a

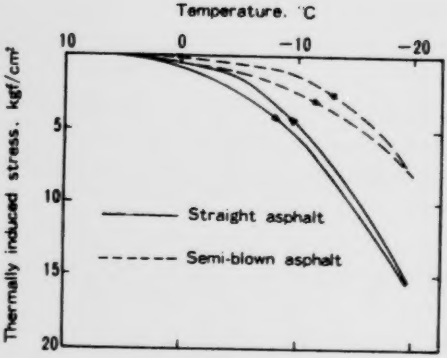

Fig.4.18 Calculated thermally induced stresses

larger stress is generated at -20°C in the mixture using straight asphalt than in that using semi-blown asphalt similar to the measured results shown in Fig.4.13.

Another simulation was performed in order to demonstrate the effects of the temperature and the temperature susceptibility of asphalt concrete. For this example, the temperature was assumed to drop straight from the initial temperature T_0°C to the final temperature (T_0-30)°C in 30 minutes. The calculated stresses at (T_0-30)°C are given in Fig.4.19 as a function of T_0. In the calculation process, a complete restraint of contraction was assumed for the asphalt concrete. In Fig.4.19, it is confirmed that thermally induced stress drops rapidly with an increase in T_0 even for the same gradient or amount of temperature change. Furthermore, it is clear that the thermally induced stresses in the asphalt concrete using straight asphalt are lower than in that using semi-blown asphalt in a higher temperature range due to the effect of temperature susceptibility.

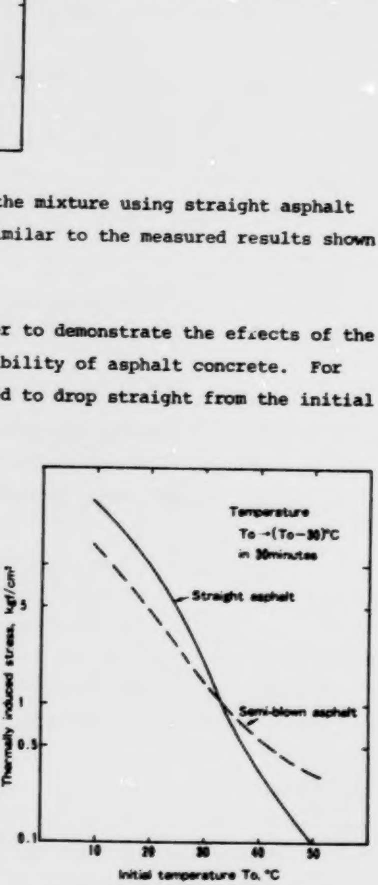

Fig.4.19 Effects of initial temperature and type of asphalt on thermally induced stresses

Because the procedure employed here can simulate thermally induced stresses in asphalt concrete as shown above, this method seems to be useful in estimating the effects of hardening or aging of asphalt on the intensity of thermally induced stresses if proper relaxation moduli can be given. However, stresses due to restraint of curling or warping of the asphalt concrete layer might have to be considered when the temperature gradient in the vertical direction is large.

4.4 Effects of repeated thermally induced stresses on mechanical properties of asphalt concrete

Fig.4.20 shows the effects of repeated thermal change up to 100 cycles on induced stresses in the specimens before curing. It is apparent that the repetition of thermal change gradually reduces the induced stresses for all mixtures. Results obtained for NS are rearranged in Fig.4.21 taking into account the effects of curing time. Fig.4.21 also shows that the thermally induced stresses seem to decrease with the number of cycles of thermal change in the same manner as shown in Fig.4.20 for all curing times.

Flexural strength and strain at failure of the specimens before and after 100 cycles of thermally induced stress following several weeks of curing are shown in Fig.4.22 and Fig.4.23. These figures reveal that the 100 cycles of thermally induced stress decrease flexural strength and increase strain at failure.

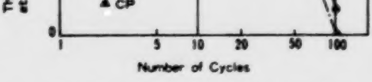

Fig.4.20 Influence of the number of thermal cycles on thermally induced stresses

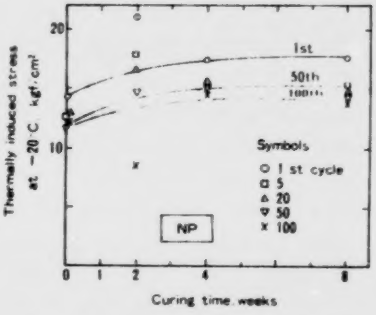

Fig.4.21 Influence of the number of thermal cycles and curing time on thermally induced stresses

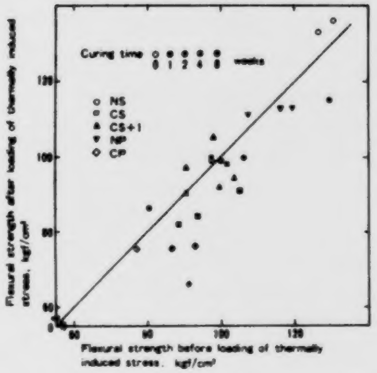

Fig.4.22 Comparison of flexural strength before and after 100 cycles of thermally induced stresses

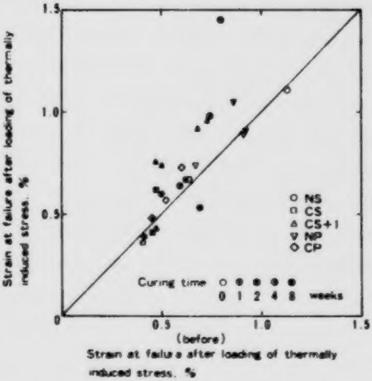

Fig.4.23 Comparison of strain at failure before and after 100 cycles of thermally induced stresses

As mentioned in this subsection, repeated thermal change or repeated thermally induced stress seems to have the effect of softening the asphalt concrete. In order to discuss this problem in detail, however, more experiments covering a wide range of test conditions are necessary.

4.5 Possibility of the occurrence of thermally induced cracking

In this investigation, any asphalt concrete without heat curing did not indicate tensile cracking under 100 cycles of thermal change from +10°C to -20°C. On the other hand, some asphalt concrete with heat curing indicated failure phenomena under the repetitions of thermal change, since the asphalt concrete became harder and the thermally induced stresses became larger.

Judging from these series of investigations, it appears that the age-hardening of asphalt concrete might have a large effect on the occurrence of thermally induced cracking in asphalt concrete. The hardening of asphalt concrete due to heating or oxidation makes the asphalt concrete brittle and susceptible to failure, and in addition, the thermally induced stresses are larger. As a result, some thermally induced cracking might occur in the asphalt concrete.

However, the prevention of such hardening of asphalt concrete might decrease the degree of occurrence of thermally induced cracking. The test results on the asphalt concrete with asphalt content of OAC + 1% shows that the degree of hardening of the asphalt concrete is less in comparison with that of the asphalt concrete with OAC, and that the thermally induced stresses in the former are also less than those in the latter. The reason why thermally induced stresses slightly decrease with the increase of asphalt content is that the effect of the decrease of relaxation moduli with the increase of asphalt content may be larger than that of the increase of the coefficient of thermal expansion. On the basis of these results, it appears that a higher asphalt content within a certain limit might delay the progressing of hardening of asphalt concrete and decrease the degree of occurrence of thermally induced cracking.

5. Conclusion

The effects of the aging or hardening of asphalt concrete by means of heat curing (70°C) on thermally induced stresses with thermal cycling (+10°C to -20°C) were investigated, and the influence of the hardening of asphalt concrete on the occurrence of thermally induced cracks in asphalt concrete was made clear. The main conclusions based on this investigation are as follows.

(1) Asphalt concrete becomes hard and brittle during the process of heat curing. The strength increases, the strain at failure decreases, and the stiffness increases. Moreover, the transition point which is the peak point of the flexural strength-temperature curve becomes higher. Similarly, the failure envelope moves to the left over the range higher than 5% of strain at failure.

(2) The tendency mentioned in (1) is more noticeable when a highly absorptive aggregate is used.

(3) The asphalt extracted from the mixes which have undergone heat curing has hardened and become brittle, judging from the penetration, softening point, penetration index and chemical composition.

(4) The thermally induced stresses of asphalt concrete at -20°C increase with curing time.

(5) The higher within a certain degree the asphalt content is, the less the thermally induced stress is. The reason may be that the effect of the decrease of relaxation moduli with the increase of asphalt content is higher than that of the increase of the coefficient of thermal expansion.

(6) A strong relationship has been observed between the thermally induced stress of asphalt concrete at -20°C and the softening point of the asphalt in the asphalt concrete.

(7) The thermal stresses induced in asphalt concrete may be estimated using the relaxation modulus and coefficient of thermal expansion of the asphalt concrete at a given temperature with noticeable accuracy.

(8) The repetitions of thermally induced stresses on asphalt concrete produce a decrease in flexural strength and an increase in the strain at failure.

(9) It is thought that the aging or hardening of asphalt concrete might have a large effect on the occurrence of thermally induced cracks, judging from the results of the investigation of actual conditions of airport asphalt pavements in Japan and the series of laboratory tests.

(10) If the hardening of asphalt concrete can be prevented, the occurrence of thermally induced cracks will be decreased. One method of preventing the hardening of asphalt concrete is to raise the asphalt content. If the level of asphalt content is raised to a suitable degree, thermally induced cracking may be minimized.

Session 2

Design and Materials Considerations for Cold Climate Pavements

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Abstract

Design and Materials Considerations for Cold Climate Pavements

This paper presents an overview of design and materials considerations required for the construction of serviceable pavement in the Province of Manitoba. Ambient temperatures in Manitoba range from +38°C to -43°C, with annual total precipitation equal to approximately 500 mm of rainfall and we experience between 1700 and 3600 degree days of frost (in Celsius). Soil types range from a CH[(A-7-5)20] to granular and both well drained and poorly drained land is encountered. Frost penetrations beneath pavements vary from 2.0 to 4.3 m in depth.

As a result of these conditions, structure design and material selection is influenced by the need to maintain low temperature pavement cracking to a minimum, the direction of some efforts towards reduction of reflection cracking and the prevention or minimizing of frost heave.

As a result of research in the late 50s and early 60s we saw the introduction of soft (SC-3000) asphaltic binders to reduce low temperature transverse cracking. As well, an experimental project in 1958 introduced the first use of foam plastic insulation beneath a pavement. About the same time, the use of a layer of granular base material between existing and new pavements was introduced both to reduce thickness of asphalt pavement overlays and to attempt to eliminate or reduce reflection cracking.

The slow curing asphalt binder successfully eliminated thermal cracking for approximately 10 years after which time the aging of these pavements provided conditions which allowed considerable cracking although without the normally experienced decrease in riding comfort. Because of changes in oil reserves the use of asphalt binders which had been cut back with potential fuels was drastically reduced. This did not pose a problem, however, as the St. Anne test road experiment had provided definitive information concerning the role of the asphalt binder relative to thermal cracking and provided the data necessary for the selection of suitable asphalt cements which would reduce thermal cracking potential.

The selection of asphalt cements of higher penetration value and/or higher viscosity characteristics coupled with the use of the gravel base sandwich design has greatly reduced the thermal cracking problem. Experiments incorporating coarse seal coats between old and overlay pavements and some experiments with membranes were also undertaken.

A description of the foamed plastic tests section results is included as well as a description of the insulation of potentially frost heaving soils and of differentially heaving culvert installations.

DESIGN AND MATERIALS CONSIDERATIONS FOR COLD CLIMATE PAVEMENTS

by

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Introduction

This paper presents an overview of design and material considerations required for the construction of serviceable pavements in the Province of Manitoba, Canada. Ambient temperatures in the Province range from +38°C to -43°C in the southern portions of the province and +16°C to -45°C in the northern portion. The normal annual total precipitation, both rain and snow, is equal to approximately 500 mm and there are from 1700 to 3600 degree days of frost (in celsius) experienced. Native soils range from granular to highly plastic clays in both the well drained and poorly drained condition. Muskeg and rock are also encountered and in the northern portions of the province there is both discontinuous and continuous permafrost.

The depth of frost penetration beneath pavements in the southern part of the province ranges from 2 metres to 3 metres, with depths of frost in the northern areas reaching in excess of 4.3 metres. The frost penetration in any given year or location depends upon moisture content of the subgrade soils, the nature of the soils, depth of snow cover and snow clearing activities. Snow fall, and therefore snow cover have been relatively light during the last three winters and rainfall during the summers has varied from extremely low to high. The areas of lower rainfall have therefore been subjected to greater frost penetration which, in some cases, has reached water bearing and frost susceptible soil layers creating frost heave, pavement cracking, and subsequent spring strength loss not previously experienced.

Traffic conditions in Manitoba vary from quite light, under 200 vehicles per day, to moderate volumes of up to 10,000 vehicles on an annual average daily traffic basis. On the average, 10% trucks traverse these roads with a few exceptions near the industrialized area of Winnipeg, in the vicinity of gravel pits and grain elevators, with recreation areas experiencing the the opposite situation where automobile traffic makes up the majority of volume. Legal axle loads in Manitoba are 9 100 kg for a single axle and 16 000 kg for a tandem axle with the provision that sufficient width of tire be provided so that the loading does not exceed 90 kg per 10 mm of tire width, on the Class A designated portion of the provincial trunk highway and Provincial road system. The Class B system of roads allows 8 200 kg on a single axle and 14 500 kg on a tandem.

The principle industries in the province are mining, fishing and forestry on a province-wide basis with agriculture and manufacturing carried on in the more southern portions. Allowable loads on the main heavily trafficked routes, which includes all concrete pavements and the higher standard asphalt surfaces, are not restricted as to spring loading conditions. All other surfaced roadways are restricted for approximately six weeks during the spring thaw, on the basis of Benkelman beam test results. Benkelman beam readings, obtained with a 9 100 kg axle load, in excess of 200 thousandths of a cm require that loads be reduced to one half the legal limit or 45 kg per 10 mm of tire width and readings of 125 thousandths of a cm up to 200 require load restriction to 60 kg for each 10 mm of tire width. The enforcement of spring load reductions has successfully provided a means whereby fairly thin pavement structures have been enabled to survive for many years without excessive maintenance costs.

Pavement Design

Asphalt pavements are designed on the basis of a relationship between the subgrade soil group index and traffic volume (1,2). Figure 1 is a set of curves used for determination of gravel base course depth versus soil group index, for asphalt surface 75 mm and 100 mm thick hot mix asphalt surfaces.

The group index of the soil is derived from an early AASHO soil classification method (3) and is determined from the following equation:

$$\text{Group Index} = 0.2\,a + 0.005\,ac + 0.01\,bd$$

a = -200 greater than 35 but less than 75 (1 - 40)
b = -200 greater than 15 but less than 55 (1 - 40)
c = liquid limit - 40, limit less than 60 (1 - 20)
d = plasticity index - 10, less than 30 (1 - 20)

Group Index maximum = 20

The depth of base determined is corrected for traffic volume as follows:

Traffic Volume	% of Standard Design
under 200 AADT	50
200-800	66 2/3
800-2000	75
2000+	100

The traffic volumes used for the design are two times the existing traffic at the time of the design. If percent trucks identified in traffic surveys exceeds 10% and no other information on weights is available, the next higher category of design standard percentage is used. Roadways whose traffic times the factor of two exceeds the 2000 figure substantially and which carry more than 10% trucks are given special attention.

Since heavily trafficked roadways have generally been the subject of vehicle weight studies it is possible for the designer to use the Shell Canada Limited (4), the Asphalt Institute method (5), the AASHTO (6) or other references to determine what increases in a structure are warranted.

It should be noted that weak, highly plastic subgrades, heavy traffic, some limitations on aggregate availability, proximity to two portland cement manufacturing plants, and 50 years of experience have resulted in the major routes in the vicinity of Winnipeg being constructed with concrete pavement. Most of the concrete is 200 mm thick with 100 mm of gravel base course and incorporating steel bar mats to resist temperature stresses. There are also several miles of 150 mm thick unreinforced concrete, a few miles of 200 mm plain pavement with dowel assemblies at transverse joints, and approximately 25 km of 10 inch unreinforced concrete pavement in the Red River Valley area surrounding Winnipeg.

Rural highways are designed to provide a finished surface approximately 1 m above prairie level to reduce snow drifting and the embankment is constructed with soil removed from side ditches. Wherever possible drainage is provided to prevent accumulations of water in these ditches. During construction, frost susceptible and organic materials are generally removed. The organic soils are used on the side and back slopes to promote the growth of vegetation and the frost susceptible soils are either wasted, blended with other materials being placed in the subgrade or placed on top of impervious soil to reduce or eliminate the ingress of water to the soils. In areas where it may be impossible to provide adequate drainage, the depth of gravel base course used is increased by 25 - 50% of the design thickness. If a frost susceptible material is encountered in a subgrade and is not to be removed, the depth of gravel base is increased by 50%. The additional base depths are provided to decrease the load distributed from the traffic to these weaker soils.

A minimum of 150 mm of gravel base course is required for all types of asphalt surface. Experience throughout the province indicated that this depth was necessary for the satisfactory performance of thin surface treatments. It had also been observed that cracked pavements on clay grades which had gravel base depths in excess of 150 mm did not exhibit the formation of bumps or ridges at the transverse cracks. These bumps were formed by the entrance of moisture through the crack in the asphalt surface causing the subsoil to swell. The gravel base layer provided a means of distributing such moisture over a larger area so that its effect on the subgrade was not localized to the vicinity of the crack.

Asphalt surface treatments are generally used, as part of the stage construction, on roadways which carry in less than 800 vehicles per day. Excellent service has been achieved with this type of construction, with many sections only requiring a reseal treatment after five to ten years to maintain the integrity of the surface. Except for cases where traffic volumes increase substantially or truck traffic volumes become excessive, it appears possible to provide service for more than 25 years without major repairs or reconstruction being required. Failures which have occurred infrequently have, without exception, been attributable to unexpected high volumes of trucks hauling from gravel pits, due to increased truck traffic associated with railway abandonment and new grain elevator terminals or, and this is extremely important, due to the use of marginal base materials immediately beneath the asphalt surface treatment. Since the surface treatment itself cannot distribute loads to any significant degree, it is of the utmost importance that the upper layers of the gravel base course be made of high quality granular materials.

In Manitoba we have been very successful employing clay-bound gravel base and, in the last fifteen years, gravel base bound with small amounts of dilute emulsified asphalt.

Many agencies would be shocked by the use of clay to bind gravel base course but we have experienced very few problems with this material. Specifications for the gravel base course layer are given in Table 1. In addition to these requirements the fraction passing the #40 sieve is tested for plasticity index. During production of the gravel base course if the material is non plastic the contractor is required to add clay binder to achieve a plasticity index not greater than 6. If the naturally produced material has a plasticity index less than one he is allowed to add clay binder to increase the plasticity index to 6, but if he is able to maintain the gravel base under the action of traffic prior to priming and paving he is not forced to add any binder material. If the plasticity index is greater than 6 the contractor is required to add supplementary granular material or lime in order to reduce the plasticity. The binder material is frequently a bid item and is required to have 75% passing a #40 sieve which material shall have a liquid limit not greater than 45 and plasticity index of not less than 10. This material is added at the time of crushing so that some mixing is achieved.

One problem that does occur with this material is due to the excessive addition of moisture during compaction which causes the binder material to float to the surface. When the binder material dries it presents an extremely hard and dusty surface which also does not allow the asphalt prime to penetrate satisfactorily. In these cases the contractor is required to rip and relay the top few millimetres to ensure that the clay binder is properly distributed throughout the base layer.

Where practical, perhaps due to the gradation of the aggregate source or the fact that suitable binder material may not be available within reasonable haul distance, the Department provides an SS-1 Emulsified Asphalt which is then diluted approximately 10 parts to one with water and this material is applied as water for the compaction process. The quantity of emulsion incorporated is generally around seven or eight litres per tonne. Surprisingly this very small quantity of asphalt is quite effective in tying the aggregate together sufficiently to achieve compaction. It should be added that frequently the final lift of gravel base course material is treated with slightly higher quantities of dilute emulsion which does act somewhat like a prime coat.

Specifications for aggregate for asphalt paving mixes are given in Table 2. In general, asphalt pavement thicknesses employed in the province are either 75 mm or 100 mm depending on total structure requirements. Of the gravel base course portion of the structure a minimum of 4" of class A material is normally specified with the balance of the base course comprised of class C.

The specifications for the asphalt binders are given in Table 3.

A mix design is prepared for every source of aggregate and additional designs are undertaken if aggregate source or gradation changes during the project. The asphalt content is determined and specified by the Department and may be altered to reflect performance or material changes during paving.

Asphalt contents vary from 5% to 6%, air voids from 2% to 7% and 98% of 75 blow Marshall is considered desirable compaction. The Department calibrates all asphalt plants and dictates material combinations, temperatures and compaction procedures.

Probably the most significant factor in the asphalt pavement design or the concrete pavement design used in Manitoba, is that a thorough investigation is conducted of the soils to be used in construction as well as the soils in the completed sub-grade. It is possible in the Province to design for very large regions based upon general soils information however this is not done simply because the soil and grade surveys undertaken provide detailed information as to local changes as well as providing information on organic material encountered, drainage patterns and other features. The success of the pavement design system is due largely to the detailed information and an understanding of soil performance.

Frost Penetration Considerations

As stated in the introduction a considerable depth of frost penetration occurs throughout Manitoba's highway system. Removal of organic materials, soil selection and drainage improvements form the major part of the frost action defense. Reduction in allowable axle loadings as outlined previously is also important for the relatively thin pavement structures to be protected against undue failures during the spring thaw.

In most cases, frost susceptible materials, Figure 2,are removed from the sub-grade. However, where the depth of excavation is excessive or the volume of frost susceptible soil is very large, then improvements to the drainage are attempted and the susceptible layers are placed above less permeable materials to cut off water supply, the depth of gravel base is increased by up to 50% or, in some locations, polystyrene foam plastic layers are used to reduce the depth of frost penetration into the susceptible soil. In general,thickness of foam plastic from 35 to 75 mm has been found to successfully reduce frost heave to acceptable levels (7). Care must be exercised in "feathering out" the foam plastic layer so as to reduce differential boundary conditions.

Foamed plastic insulation has been also successfully employed to reduce differential movement at culvert locations. A standard two inch thick foam plastic layer is employed beneath all concrete box culverts. In troublesome areas,both sheet polystyrene foam beneath culverts and a sprayed-on urethane foam inside culverts have been employed. These techniques minimize the increase in frost penetration due to the cold air entering the culverts and depressing the frost line to a greater depth than encountered in the balance of the roadway.

The insulating approach, employing foamed plastic, was conceived in Manitoba in 1958 and a trial installation was completed in 1962 (7). It was found that 82 mm of polystyrene foam prevented frost penetration and only a few millimetres of frost penetrated a 50 mm thick insulating layer.

For determining the depths of freeze and thaw in soils and the effects of insulation methods, a computer program from the State of Alaska using a multilayer solution of the Modified Berggren Equation has been used in preliminary designs (3). Otherwise, we depend mainly on past experience or information from the insulation supplier (9).

The required input data is based on climatological data for the site (mean annual temperature, design air freezing or thawing index, and length of freezing or thawing season) and the geotechnical properties for each layer of material (dry density and moisture content and, if available, the heat capacity, thermal conductivity and latent heat of fusion properties). The insulation was placed on a thin layer of sand, used for levelling, and was covered by 300 mm of gravel base and a 100 mm asphalt surface.

In 1965 a dual multi-plate corrugated steel drainage structure was installed with styrofoam insulation. However no data was collected regarding the temperature regime beneath the road as vandals successfully destroyed the thermocouple measurement station.

The occurrences of severe differential frost heaves of up to 90 mm over four dual arch corrugated steel pipe locations during the 1976-77 winter season were minimized during the ensuing winter seasons as a result of insulating the culverts. Two methods of insulating the culverts were employed as follows:

a) a layer of Styrofoam insulation was placed beneath one pair of culverts, and

b) a layer of polyurethane insulation was sprayed on the interiors of three culvert pairs.

Thermocouple instrumentation was installed at the Styrofoam insulated site and the adjacent polyurethane insulated site for comparing the depths of frost penetration. Also, profiles were taken at each location to determine any differential heave.

Detailed site evaluations were conducted for two winter seasons. The 1977-78 winter was normal with a freezing index of 2065° days C while the 1978-79 winter was particularly severe having a freezing index of 2578° days C. The maximum depth of frost penetration was 2.13 m for both insulation types in 1977-78 compared to 2.68 m for the Styrofoam site and 2.62 m for the polyurethane site in 1978-79.

Prior to insulating, the measured differential frost heaves ranged from 52-82 mm. After insulating, the differential frost heaves ranged from 9-30 mm in 1977-78 and from 9-43 mm in 1978-79. Detrimental differential heave was significantly reduced by insulating the culverts, especially as indicated by the measurements from the 1978-79 season which was very abnormally cold.

Both the Styrofoam and the spray-on polyurethane methods were equally effective in alleviating the frost heave problem. The depth of frost penetration and the heaving were effectively equalized with the rest of the roadway; which was the intent of the project.

It is to be noted that insulated roadways present two problems. Firstly the insulation reduces heat flow from the subgrade to the surface and can result in frost remaining on the pavement. Since adjacent sections could be frost-free, a hazardous condition might result. Secondly, insulated roadways are subject to deflection year round which could result in decreased pavement life.

Pavement Overlay Design

During the spring season all surfaced roadways in the Province are tested weekly using a Benkelman beam to determine structural capacity and to provide input for the spring restriction system. The spring restrictions imposed are as follows:

Rebound Standard (cm)	Load Limit and Pavement Structure Design Levels
0.200	Loads to be restricted to 45 kg per 10 mm width of tire when Spring Deflections exceed this level.
0.125	Loads to be restricted to 60 kg per 10 mm width of tire when Spring Deflections exceed this level. Design parameter for 8 200 kg single axle, and 14 500 kg tandem axle with application of Spring Load Restrictions.
0.115	Design parameter for 9 100 kg single axle, and 16 000 kg tandem axle with application of Spring Load Restrictions.
0.090	Design parameter for 8 200 kg single axle, and 14 500 kg tandem axle with no Spring Load Restrictions (90 kg per 10 mm).
0.080	Design parameter for 9 100 kg single axle, and 16 000 kg tandem axle with no Spring Load Restrictions.

A combination of fall panel ratings, spring restriction requirements and maintenance costs are used to determine which pavements require upgrading.

The procedure employed to determine the structural requirements follows the CGRA (now RTAC) (10) procedure employing the thickening curves shown in Figure 3.

Many overlay designs are relatively thin and present practice is to use straight bituminous overlay if the additional thickness does not exceed 100 mm requirement to reduce spring rebound values to acceptable levels.

Straight bituminous overlays seldom exceed 125 mm in thickness but where greater thicknesses are required for strength development purposes a "sandwich" design is employed whereby a gravel layer is placed on top of the existing surface and then followed with 75 - 100 mm of new asphaltic pavement. The gravel base interlayer must exceed 100 mm thickness for practical laying and compaction purposes.

It is found that this gravel base layer does reduce reflection cracking and so far in Manitoba has not resulted in any failure conditions developing due to accumulations of moisture in this semi-permeable layer. This is perhaps due to adequate cross fall on the existing surface.

It is also our practice, for cases where less than 150 mm of gravel base course are supporting the existing pavement, to remove the existing pavement and recycle it either as fill, base or, in the future as a new bituminous surface.

Asphalt Surface Treatments

For fairly lightly trafficked roads, an asphalt surface treatment is frequently employed directly on the gravel base course layer. In most cases this gravel base course is equivalent to 75% of what would have been required to support a 75 mm asphalt surface. The design criteria was developed from experience which dictated that a minimum of 150 mm of gravel base course be applied with other depths determined from the design curve.

The surface consists simply of a gravel base course primed with either MC-30 asphalt or with emulsified asphalt which is allowed to cure or is lightly sanded. Following the priming, an MC-500 or high float emulsified asphalt is applied followed by sanding or a crushed aggregate application.

Except for unusual traffic conditions, such as gravel haul routes or routes leading to large grain elevators, this type of dust free surface has provided excellent service in the Province.

One of the secrets of success of the comparatively thin surface structures throughout the province, including those with hot mix surfacing, is the close attention to reduction of allowable loads during the spring thaw.

Cold Weather Surface Considerations

Over 35 years ago the problem of thermal or low temperature cracking of asphalt pavement was not too evident partly due to low traffic volumes and partly due to the types of asphalt binders employed. In the early days SC6 asphalts were employed quite widely. The nature of the asphalts was such that low temperature cracking was not much of a problem. However a combination of a switch in asphalt binders to harder, low penetration, asphalt cements increased the severity of low temperature cracking and this, coupled with thinner gravel base course layers due to small budgets and heavy construction program requirements led eventually to many areas experiencing the formation of bumps at the transverse cracks due to infiltration of moisture which passed through the thin gravel base course layers into the subgrade soils which subsequently swelled due to the moisture content increase.

A great deal has been published, particularly throughout North America, concerning this subject and a great deal of laboratory and field experimentation has been undertaken. Initially, in Manitoba, the seriousness of the transverse cracking promoted a change in asphalt binder to SC5 even though there was a potential danger of rutting under heavy traffic conditions. The SC5 binders were employed for many years quite successfully but, since these binders tend to age fairly rapidly and did develop a great deal of cracking after eight to ten years of service, although without any loss in performance, and partly because of a desire to reduce the use of cutter stock which could be used as a fuel, changes began to be made to introduce soft asphalt cements.

Asphalt cements with penetrations in the 300/400 and in the 200/300 penetration range were employed but many of these were found not to resist the low temperature cracking previously experienced.

It is well documented that the viscosity characteristics of the asphalt binder as well as its penetration level were extremely important in the provision of a non cracking asphalt layer. It has also been learned that, due to low temperature cracking being initiated at the surface (11) combinations of temperature susceptible lower layers with non temperature susceptible upper layers could be successfully employed. Manitoba asphalt cement specifications are given in Table 3.

There is still considerable controversy raging between user agencies, suppliers and some technical organizations concerning the means to specify asphalts to provide satisfactory mix stability, reduction in low temperature cracking and practical and economic use of crudes and refining processes available. A graphical representation of a specification presently under consideration by the Canadian General Standards Board is shown in Figure 4.

Reduction of Reflection Cracking

To date, in Manitoba, the combination of use of asphalt binders with low temperature susceptibility and a gravel base interlayer appears to have largely overcome the problem of low temperature cracking and reflection cracking. For cases where only a thin overlay is required, some experimentation has been undertaken using a rubber membrane at existing transverse cracks. It has been found that the rigid membranes either fail in tension or create a transverse crack offset from the original to the edge of the membrane strip. The highly flexible membranes stretch over the opening existing crack thus creating a failure in the overlay. There is an advantage, however, in that these latter membranes do seal out moisture from the old pavement structure.

Another experiment was undertaken employing a coarse aggregate seal coat between the old pavement and the new overlay. This procedure seems to have been marginally effective following one winter but no conclusions can be drawn at this time.

It is anticipated that the development of recycling projects and equipment will alleviate the reflection cracking problem through the removal of existing cracked surfaces, the addition of softening agents and replacement of the surface. Care must be taken, however to ensure that the combination of old and new asphaltic binders results in low temperature susceptibility.

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Table 1

GRAVEL BASE COURSE SPECIFICATION

Aggregate and supplementary granular material shall consist of sound durable particles of crushed rock, gravel, stone, sand and fines free from sod, roots and organic material.

The crush count is defined as the percentage by weight of aggregate particles retained on a No. 4 sieve which are not shale and which have at least one freshly fractured face.

The Los Angeles abrasion loss and gradation analysis will be performed using A.S.T.M. test procedures.

Shale Content is defined as the percent by weight of the particles retained on a No. 4 sieve that are shale particles.

The aggregate shall be well graded and shall not vary from maximum to minimum of the specification ranges for consecutive sieve sizes. The gradation and physical requirements for each class shall be as follows:

GRANULAR BASE COURSE

	CLASS "A"		CLASS "B"	CLASS "C"	
Passing Standard Sieves	Gravel	Limestone	Gravel or Limestone	Gravel	Limestone
2" sieve					100%
1 1/2" sieve				100%	
3/4" sieve	100%	100%	100%		
5/8" sieve	80-100%				
No. 4 sieve	40- 70%	35- 70%	30- 75%	25- 80%	25- 80%
No. 10 sieve	25- 55%		25- 65%		
No. 40 sieve	15- 30%	10- 30%	15- 35%	15- 40%	
No. 200 sieve	8- 15%	6- 17%	4- 18%	*4- 20%	5- 20%
Minimum Crush Count	35%	—	25%	—	—
Maximum Los Angeles Abrasion Loss	35%	35%	35%	35%	35%
Maximum Shale Content	12%	—	12%	12%	—

* Where the Contractor is supplying, but not constructing, Gravel Base Course Class "C" the minimum passing the 200 sieve shall be 8%.

Table 2

ASPHALT PAVING MIX AGGREGATE

The gradation of the aggregate during production shall be controlled to ensure that the V.M.A. of the bituminous paving material falls between 14 percent and 17 percent, based on a 75 blow Marshall design method.

The gradation of the final blended aggregate for bituminous pavement will be as specified by the Engineer, based on the mix design, and shall fall within the following gradation requirements.

Passing 3/4 inch sieve	100%
Passing 5/8 inch sieve	95 - 100%
Passing 3/8 inch sieve	70 - 90%
Passing No. 4 sieve	50 - 70%
Passing No. 10 sieve	35 - 55%
Passing No. 40 sieve	17 - 29%
Passing No. 20 sieve	3 - 7%

When differently graded aggregates obtained from one or more sources are to be blended to form one aggregate, they shall be added uniformly and fed separately to the crusher using controlled belt feeders.

Unless otherwise specified in the Special Provisions, the Contractor shall separate aggregate for bituminous pavement into a coarse fraction and a fine fraction. The separation shall occur at the time of production unless a delayed separation is authorized in writing by the Engineer.

The coarse fraction shall be any material retained on a No. 4 sieve. Aggregate separated in this manner will be referred to as split aggregate and the stockpiles of each material as split aggregate stockpiles.

The Engineer will establish gradation ranges to govern the production of both the coarse and fine fractions. The gradation ranges will apply to each sieve size and will be determined by applying the following limits to the average gradation:

Percent passing a 5/8 inch, 3/8 inch and a No. 4 sieve	±6
Percent passing a No. 10 and No. 40 sieve	±5
Percent passing a No. 200 sieve	±2

Table 3

PROVINCE OF MANITOBA SPECIFICATION FOR ASPHALT CEMENTS (JAN. 1979)

<table>
<tr><th></th><th colspan="2">Penetration Graded @ 25°C</th><th colspan="2">Viscosity Graded @ 60°C</th></tr>
<tr><th>Type</th><th>150-200</th><th>150-200 H</th><th>A.C. 20</th><th>A.C. 27.5</th></tr>
<tr><td>Penetration @ 25°C (ASTM D 5)</td><td></td><td></td><td colspan="2">Viscosity & Penetration must fall within the straight line envelope bounded by the following points.<table>
<tr><th>Point</th><th>Vis.</th><th>Pen</th><th>Point</th><th>Vis.</th><th>Pen</th></tr>
<tr><td>A</td><td>20</td><td>300</td><td>A</td><td>27.5</td><td>300</td></tr>
<tr><td>B</td><td>23</td><td>350</td><td>B</td><td>31</td><td>370</td></tr>
<tr><td>C</td><td>40</td><td>250</td><td>C</td><td>57</td><td>270</td></tr>
<tr><td>D</td><td>32</td><td>220</td><td>D</td><td>50</td><td>200</td></tr>
</table>Viscosity @ 60°C in Pascal Sec. (ASTM D 2171)
Penetration @ 25°C in dmm using 100 gm 5 sec. (ASTM D 5)</td></tr>
<tr><td>Viscosity @ 135°C Min. (ASTM D 2170)</td><td>195</td><td>250</td><td>125</td><td>150</td></tr>
<tr><td>Flash Point in Deg. C (ASTM D 92) min.</td><td>220°</td><td>220°</td><td>175</td><td>175</td></tr>
<tr><td>Solubility in Trichloroethylene (ASTM D 2042) min.</td><td>99.5%</td><td>99.5%</td><td>99.5%</td><td>99.5%</td></tr>
<tr><td>Ductility in mm (ASTM D 113) min.</td><td>1000*</td><td>1000*</td><td>1000*</td><td>1000*</td></tr>
<tr><td>Loss on Heating (ASTM D 6) max.</td><td>1.0%</td><td>1.0%</td><td>—</td><td>—</td></tr>
<tr><td>Penetration after loss on Heating (ASTM D 5) % of original</td><td>70</td><td>70</td><td>—</td><td>—</td></tr>
<tr><td>Ratio of Absolute Viscosity of Residue from T.F.O.T. to original (ASTM D 1754 & D 2171)</td><td>—</td><td>—</td><td>4.0 max.</td><td>4.0 max.</td></tr>
</table>

*If Ductility is less than 100 material will be acceptable if ductility is 100 or better at 15.5°C (60°F).

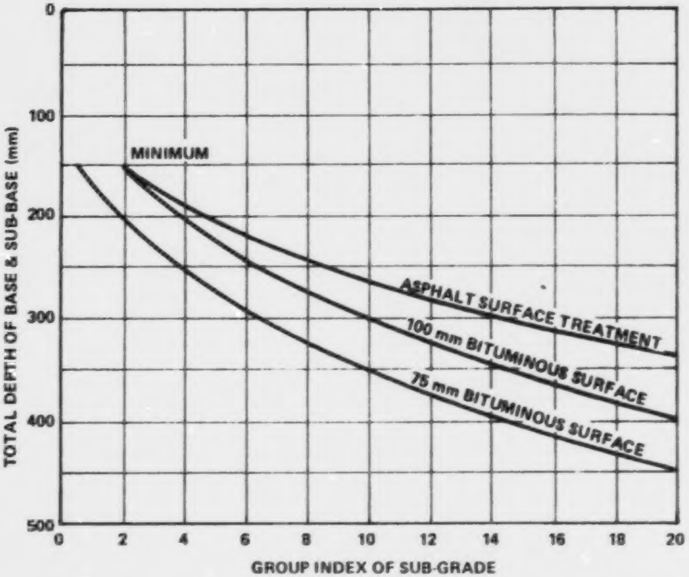


Figure 1/ Design Chart for Depth of Granular Base Under Bituminous Surfaces

Determination of frost susceptibility

A soil sample is considered as frost susceptible if the following conditions are satisfied.

Let

SI = % silt in sample
CL = % clay in sample
FS = % fine sand in sample
CA = % coarse aggregate in sample
CS = % coarse sand in sample, and
PI = plasticity Index of sample

If $(SI+CL) \geq 20$

and $PI \leq 12$
and $CL \leq 25$
and $(FS+SI) > 60$
and $(CA+CS) \leq 20$

Then the material is frost susceptible.

Figure 2/ **Frost Susceptibility Criteria (Documentation of Computer Analysis Instructions)**

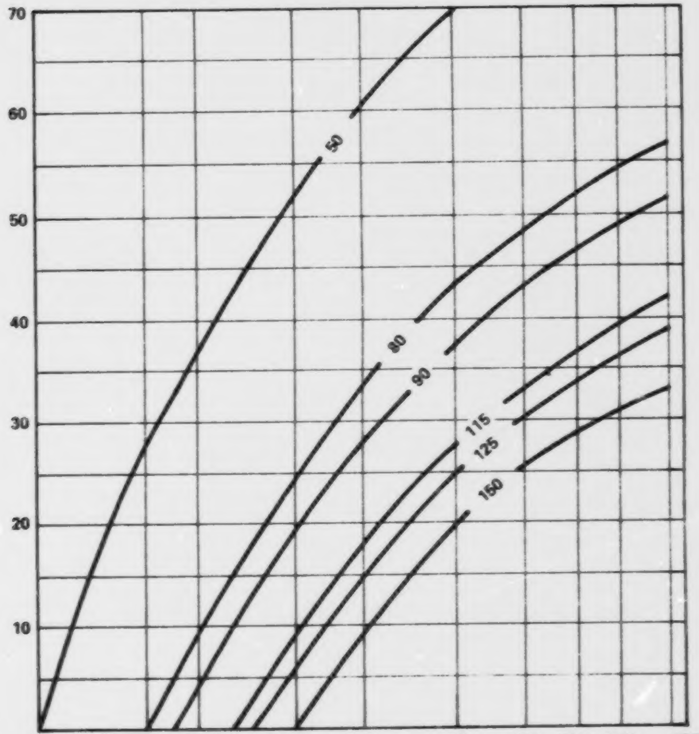
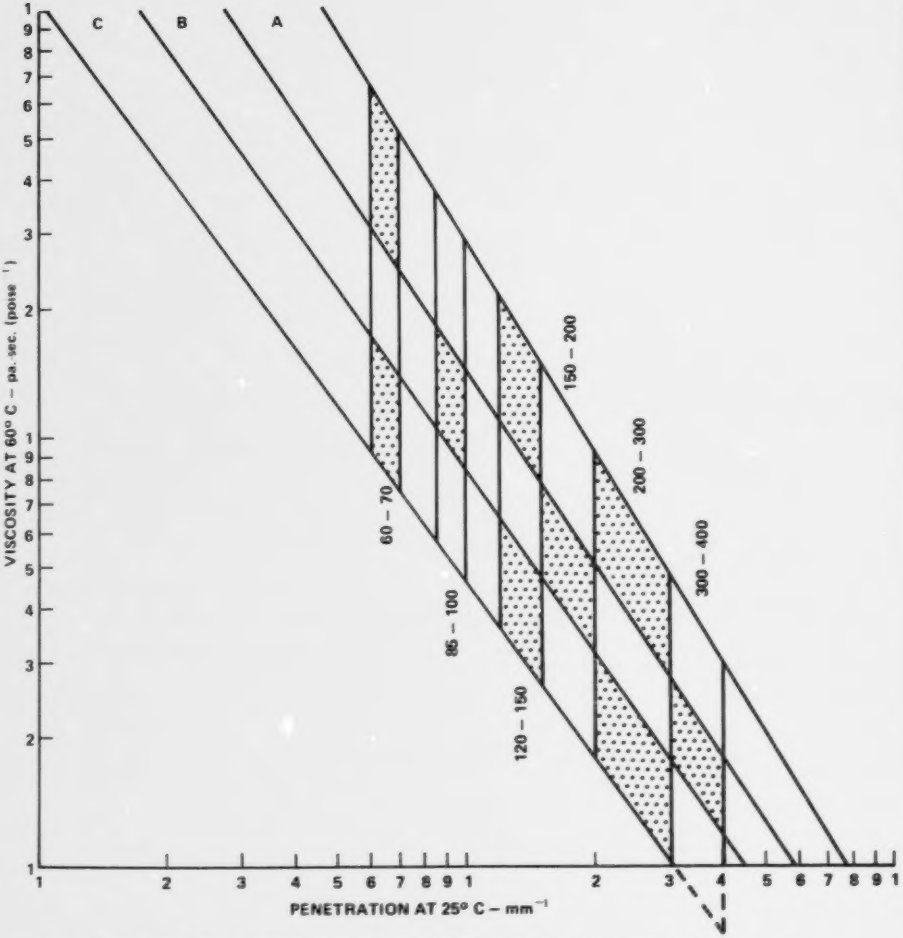


Figure 3/ Additional Depth of Granular Base Course Required to Reduce a Benkelman Beam Rebound on an Existing Surface to Various Designated Rebound Values



Note: Types A to C intended to provide asphalts which exhibit low (Type A), intermediate (Type B) and high (Type C) temperature susceptibility.

Figure 4/ Graphical Representation of Penetration-Viscosity Relationships Being Considered for Canadian Use (Preliminary Only)

Session 2

Design, Construction and Maintenance of Asphalt Pavement for Expressways in Cold Areas

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DESIGN, CONSTRUCTION AND MAINTENANCE OF ASPHALT PAVEMENT FOR EXPRESSWAYS IN COLD AREAS

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ABSTRACT

Main requirements for surface course mixtures for expressways in cold areas are wear resistance, skid resistance, and anti-flow characteristics in summer. In Nihon Doro Kodan, to develop appropriate surface mixtures satisfying these characteristics, a series of laboratory tests has been conducted with a traffic simulator and several test pavement sections have been constructed to evaluate the laboratory test results. As a result of these tests, mix design standards for the surface course in cold areas are provided. In the standards, a dense-graded asphalt concrete is adopted and maximum size, gradation and asphalt content are specified.

The materials of pavement and subgrade within the depth of frost penetration have been selected as unsusceptible to frost.

The surface mixtures for cold areas contain less coarse aggregates, and a little more filler and asphalt than those in warm areas.

Regarding surface conditions, rutting has generally become a serious problem on expressways. Especially in cold areas, the increment of rutting by studded tires and tire chains in winter is found to be four

times greater than that in warm areas and frequency of rehabilitation is twice that in warm areas.

With the extension of expressways in cold areas, the frequency of rehabilitation is anticipated to increase and the expenditure of rehabilitation will become a serious problem. Recycling of pavement is expected to be effective and an experimental project has been carried out to reduce the expenditure of maintenance and rehabilitiation.

1/PREFACE

Ever since a section of the Meishin Expressway (Nagoya-Kobe) was opened to traffic in 1963 as the first expressway in Japan, the total length of the expressways in operation has increased to reach 3,010 km as of March, 1982. It is planned, by law, to complete the nationwide expressway networks covering 7,600 km, and the length of the expressways in operation is only 40% of the planned length. However, the expressways in operation are running through the most populous part of the Japan Archipelago and the main part of the expressway networks may be said to have been completed. Ninety-six percent of expressways has been paved with asphalt concrete because of economical construction cost and ease of rehabilitation.

The traffic volume of the expressways ranges from several thousand to the order of 110,000 vehicles per day, changing from place to place. The climate of Japan changes considerably depending on the location and season and is characterized by heavy rain and snowfalls. Therefore, Japan has striking climatic variations ranging from sub-arctic to subtropical. Since the traffic and climatic conditions, which have a great influence on asphalt pavement, change from place to place, it is necessary to plan carefully the design, construction and maintenance of pavement.

In this paper we describe how Nihon Doro Kodan (The Japan Highway Public Corporation) copes with design, construction and maintenance of pavement on expressways, and we compare pavement in cold areas with that in moderate areas.

2/ OUTLINE OF EXPRESSWAYS IN JAPAN

The roads in Japan are classified into four classes by law:

- national expressways,
- national highways,
- municipal roads, and
- town and city roads.

The national expressways are the roads which make up the cardinal portion of the nationwide automobile traffic network. All of the national expressways employ the toll system, and their structural standards are very high as understood from the divided highway structure with full access control. The national expressways are planned and constructed to serve as trunk roads for aiding national land development in line with Government policies, economical activities and various other projects. Construction and operation of such national expressways is the responsibility of Nihon Doro Kodan, which was established in 1956 under the supervision of the Minister of Construction.

The first expressway in Japan was the Meishin Expressway and its first section was opened to traffic in 1963. This was followed by construction of the Tomei Expressway (Tokyo-Nagoya) and the Chuo Expressway (Tokyo-Fujiyoshida). Other arterial expressways, running through the entire Japan Archipelago from Hokkaido to Kyushu, were contructed subsequently and opened to traffic. As of March, 1982, the total length of 3,010 km of national expressways was already opened to traffic and sections totalling 2,405 km were under construction. The present situation and future projects of expressways are shown in Figure 1. According to the national expressway network construction program, it is planned to build expressways totalling about 7,600 km in the future. So, only 40% of the planned length is in operation. Since expressways to be constructed hereafter cross the Japan Archipelago and run through mountainous regions, topographic and climatic conditions will make construction more difficult.

Distribution of annual average daily traffic (AADT) between two successive interchanges in 1980 is shown in Figure 2. As Figure 2 shows, AADT ranges from several thousand to 110,000 vehicles per day with an average of 23,000 vehicles per day. Hereafter, in this paper, the warm areas are included in the moderate areas. The shadowed portion in Figure 2 illustrates the AADT of the Tomei and the Meishin Expressways which are in the moderate areas. The two expressways are arterial roads connecting Tokyo, the capital of Japan, and Osaka, the second largest city in Japan, so the traffic is extraordinarily heavy. On the other hand, there is little traffic on expressways in cold areas.

The climate in Japan changes considerably from place to place. Japan is situated on the eastern side of the Eurasian continent and has complex topographical features. For this reason, the climate in Japan is characterized by heavy rain and snow and marked changes in temperature in different seasons and districts as shown in Figure 1. The regions through which the expressways pass range from the snowy and cold regions, with maximum snow depth reaching 280 cm, to the sub-tropical regions with the temperature of asphalt pavement surface sometimes exceeding 60°C (140°F).

3/ PAVEMENT DESIGN IN COLD AREAS

The first expressway constructed in a cold area was the Doo Expressway (Sapporo-Chitose section) which was opened to traffic in December, 1971. The problems met at the stage of design were as follows. The problems associated with the structural design of the pavement were the prediction method of the depth of frost penetration and the amount of frost heave, and the estimation of loss of subgrade bearing capacity during the frost-melting period. The problem in the mix design of asphalt concrete was abrasion of the surface course due to studded tires and chained tires. The important points in the design of the cross-section were as follows: the snow should be removed efficiently and quickly from the road surface, and the melted snow that piled on the shoulder and the median should not flow to the roadway.

In Nihon Doro Kodan, to cope with these problems, a series of laboratory tests and field studies was conducted. The standards and the way of approaching these problems are put together as a part of the Design

Manual of Nihon Doro Kodan taking into account the results of studies and standards of other organizations. The outline is described below.

3.1/ Structural Design

In the thickness design of asphalt pavement, the necessary pavement thickness is obtained from the total number of equivalent 10-ton axle loads in one direction during a 10-year period and the design CBR value of the subgrade. This method has been developed from the results of the AASHO Road Test with modifications made in the light of actual experience in Japan.

With expressways in areas susceptible to frost action, if the frost penetration is greater than the total thickness of the pavement, the materials of the subgrade within the depth of frost penetration are to be replaced with materials unsusceptible to frost. The standards of replacement are as follows [1].

- Material

  Maximum size: 50 mm; weight percent passing 4.76 sieve: 30% - 70%; weight percent passing 0.074 mm sieve: 10% or less; Plasticity Index for the fraction passing 0.42 mm sieve: 6 or less; Modified CBR: 10% or more.
- Depth of Soil Replacement

  The depth of soil replacement is determined, in principle, from the depth of frost penetration observed at the site. However, the depth of soil replacement is determined by established experience, if current. If there is no current experience data and if it is impossible to observe at the site, the depth of soil replacement is determined by estimation based on meteorological data and is obtained by means of Equation 1:

  $$Z = C\sqrt{F} \qquad (1)$$

  where, Z = the depth of frost penetration (cm)

  C = constant

  F = freezing index (°C-day)

The value of C varies with the construction field condition and it is usually taken as 4. The freezing index is the absolute value of the sum of the products of monthly average temperatures which fall below 0°C, multiplied by the total number of days in the respective months.

3.2/ Mix Design

3.2.1/ Study of Wear-Resistant Asphalt Mixture

The main requirement for surface course mixtures in cold areas is wear resistance. Since the temperature of asphalt pavement surface reaches about 60°C (140°F) in summer even in cold areas in Japan, stability at high temperatures is also required. In Nihon Doro Kodan, in order to set a standard of mix design for asphalt surface mixtures satisfying these characteristics, a series of laboratory tests has been conducted with a traffic simulator (Photo 1) and several test pavement sections have been constructed to evaluate the laboratory test results.

3.2.1.1/ Comparison of Asphalt Mixtures to Achieve Wear-Resistant Mixtures

One of the primary functions of the surface course is to provide good skid resistance. Since the travelling speed on expressways is very fast, skid resistance at high speeds is required from the viewpoint of safety. For this reason, a dense-graded asphalt mixture has mainly been used so far. However, there was little experience of use in cold areas and wear-resistant properties of the dense-graded asphalt mixture were unknown. So a comparison test of dense-graded asphalt mixture, wear-resistant Topeka and Guss asphalt mixture has been conducted with the traffic simulator [2].

Table 1 shows the test conditons and Table 2 shows the types of asphalt mixtures compared and these gradations. Figure 3 shows the relationship between the average wear depths of mixtures and number of wheel passes. It is clear from Figure 3 that the wear resistance decreases from the Guss asphalt mixture to the wear-resistant Topeka and dense-graded asphalt mixture, and Guss asphalt mixture possesses high wear-resistant properties compared with others. However, there is no significant difference between wear-resistant Topeka and dense-graded asphalt mixture.

However, Guss asphalt mixture and wear-resistant Topeka have low stability at high temperatures [3] and low skid resistance at high speeds [4]. Finally, a dense-graded asphalt mixture was adopted for expressways.

3.2.1.2/ Examination of the Dense-Graded Asphalt Mixture

In order to improve wear-resistant properties of the dense-graded asphalt mixture, the effects of maximum size of coarse aggregates, gradation, asphalt content and others on the wear-resistant properties were studied. Main results are described below.

1/ Effect of Maximum Size of Coarse Aggregates

Wear resistance of the dense-graded asphalt mixture with maximum coarse aggregate size of 20 mm is compared with that of the conventional maximum size of 13 mm by means of the traffic simulator [5]. Figure 4 and Figure 5 show the results from chained tires and studded tires, respectively. Less pavement wear is caused by studded tires and chained tires to mixtures containing coarse aggregates with the maximum size of 20 mm. It was confirmed that the dense-graded asphalt mixture containing hard coarse aggregates with the maximum size of 20 mm possesses wear resistance as high as Guss asphalt mixture.

2/ Effect of Gradation

Figure 6 shows the effects of weight percent passing 2.38 mm sieve of dense-graded asphalt mixture with coarse aggregates with the maximum size of 20 mm on the wear resistance and the rate of deformation [6]. Figure 6 shows the composite results of two tests. The solid line in Figure 6 indicates the wear resistance recorded by the traffic simulator, whereas the broken line indicates the stability of mixtures at high temperature (60°C) by means of the wheel-tracking test. From Figure 6, the volume of wear tends to decrease as the weight percent passing 2.38 mm sieve increases. However, it was found that there is no significant difference of wear volume over 45% for 2.38 mm sieve passing. On the other hand, when the weight percent passing 2.38 mm sieve increases immoderately, the rate of deformation at high temperatures increases and the stability of the mixture is lost. Therefore, it is necessary to choose appropriate gradation according to environmental conditions.

3/ Effect of Asphalt Content

Figure 7 shows the effects of the asphalt content of the dense-graded asphalt mixture with coarse aggregates with the maximum size of 20 mm on the wear resistance and the rate of deformation [6]. From Figure 7, the volume of wear decreases linearly as the asphalt content increases. In the same way as the gradation, when the asphalt content increases immoderately, the rate of deformation at high temperatures increases. Therefore, it is also necessary to take environmental conditions into account.

4/ Other Results

Though the following are not directly concerned with the choice of materials and mix design, the results of the laboratory test with the traffic simulator are interesting, and worth noting.

Figure 8 shows the relationship between pavement surface conditions (dry or wet) and volumes of wear [7]. As in Figure 8, the wear volumes on a wet surface are 1.5 to 2.5 times larger than those on a dry surface by studded tires and 2.5 to 3.0 times those by chained tires. These results suggest that the surface condition has influence upon the volume of wear in cold areas, and the volume of wear in areas where wet surface conditions continue for a long period is very large.

Figure 9 shows the volume of wear by difference in running speeds [7]. In the case of studded tires, the volume of wear increases as the running speed increases, but by a small increment. On the other hand, in the case of chained tires, the rate of increase is large compared with that by studded tires and increases rapidly with speeds between 40 km/h and 50 km/h, in particular. This result suggests that it would be possible to decrease the volume of wear by limiting the speed of vehicles to 40 km/h or less in the case of chained tires.

As for wheel load, the volume of wear by studded tires increases linearly as the wheel load increases. In the case of chained tires, volume of wear increases as the wheel load increases, but by a small increment [5, 8].

When rearranging the study results, factors contributing to wear resistance and flow resistance are summarized in Table 3.

3.2.2/ Standard of Mix Design for Wear-Resistant Asphalt Mixtures

Nihon Doro Kodan specifies the standards of the mix design for asphalt mixtures in snowy and cold areas as in Table 4, Table 5 and Figure 10 [9, 10], based on the laboratory test results of wear-resistant asphalt mixtures mentioned in the preceding clause. By the way, the Marshall method has been widely adopted for asphalt concrete mix design in Japan. Table 5 shows the penetration grade of asphalt cement and the number of blows for the Marshall Stability Test to determine the optimum asphalt content.

3.3/ Design of Road Cross-Section

The factors in design of the road cross-section in snowy and cold areas are the shape of median strip, crossfall and drainage facilities of the shoulder and widening for depositing snow. With regard to those problems, Nihon Doro Kodan specifies the standards as follows [11].

3.3.1/ Shape of Median Strip

In order to prevent the melted snow piled on the median from flowing out to the roadway and to carry on snow removal efficently, a depressed type of median in which the surface is lower than the roadway surface is adopted as standard. An example of a median strip is shown in Figure 11.

3.3.2/ Crossfall and Drainage Facilities of Shoulder

In the case where the piled snow on the shoulder melts and often flows out to the roadway, it is prescribed to bend a part of the shoulder outside as shown in Figure 12. Drainage facilities on the shoulder are to be constructed so as to prevent the melted snow from flowing out to the roadway and/or damming up by a snow bank. Figure 13 shows an example of drainage facilities on the shoulder.

3.3.3/ Widening for Depositing Snow

The amount of widening of the shoulder for depositing snow on cut sections is determined according to snow depth as in Table 6. Since the figures in Table 6 are applied in the case of a slope gradient of 1:1 and a slope in the east or west direction, it is necessary to correct these figures in the case where the slope gradient and slope direction are different.

The shoulder on fill sections is not usually widened. In the case where a length of slope is short, however, it is standard to secure the location of a 7 m width from the top of the slope as snow removal space. On bridge sections, the shoulder is widened about 1 m or the lateral location of a 7 m to 10 m width from the edge of the bridge is secured depending upon regional conditions.

4/ PAVEMENT CONSTRUCTION IN COLD AREAS

4.1/ Pavement Types and Composition

Table 7 shows the pavement length of expressways divided by types and regions as of March, 1982. Expressways in cold areas account for 28% of the total. Asphalt pavement and Portland cement concrete (PCC) pavement are in the ratio of 96:4. The reason why the asphalt pavement was used almost exclusively for expressways is the a) low initial cost, b) suitability for paving roads in operation and short-time curing, c) possibility of staged construction and d) ease of rehabilitation.

Typical cross-sections of the asphalt and PCC pavement structures constructed in the past are shown in Figure 14. In the Meishin Expressway, the pavement basically consists of 10 cm asphalt concrete surface and binder courses, 20 cm crushed stone base and 20 cm granular subbase courses. In the Tomei Expressway, the asphalt treated mixture of 15 cm to 20 cm in thickness is used for the base course. The granular or cement treated mixture of 20 cm in standard thickness is used for the subbase course. The composition of pavement in the Tomei Expressway is being adopted as standard in the expressways being constructed after the Tomei Expressway (hereinafter referred to as "new expressways").

4.2/ Design of Wear-Resistant Asphalt Mixtures

The wear-resistant mixtures on expressways are designed and produced giving consideration to maximum size of hard coarse aggregates, gradation and asphalt content, as described in Section 3.2. The actual mixtures which were constructed will be described in this Section.

4.2.1/ Gradation

Figure 15 shows the relative frequency distribution of weight percent passing 2.38 mm sieve in the Meishin Expressway, the Tomei Expressway and new expressways. The average of weight percent passing 2.38 mm sieve is 45.9% in moderate areas of new expresssways and 48.0% in cold areas as compared with 45.8% in the Meishin Expressway and 47.3% in the Tomei Expressway. More coarse aggregate is adopted in the areas where deformation at high temperatures or skid resistance is regarded as important; less coarse aggregate is adopted in the cold or little traffic areas where wear or aging of asphalt cement is regarded as important.

As for weight percent passing 0.074 mm sieve, the amount of mineral filler (limestone dust) is specified to be 3% or more in all aggregates or 50% or more by weight in dust. Furthermore, mineral filler is used, to a large degree, for surface mixtures in cold areas in anticipation of the wear-resistant effects of filler bitumen.

4.2.2/ Asphalt Content

Relative frequency distribution of asphalt content is shown in Figure 16. Average asphalt content is 6.08% in moderate areas of new expressways and 6.27% in cold areas as compared with 6.15% in the Meishin Expressway and 6.39% in the Tomei Expressway. Optimum asphalt content in new expressways is selected based on the Marshall Stability Test and penetration grade of asphalt cement but with a modified number of blows corresponding with regional and traffic conditions, and on experiences of construction and maintenance of the Meishin and the Tomei Expressways. As a result, average asphalt content in cold areas is about 0.2% more than in moderate areas.

4.2.3/ Marshall Test Values

Marshall stability and Marshall stiffness (stability/flow value: obtained from the Marshall Test and its dimension is [100 kg/cm]) of surface mixtures used in the Meishin Expressway, the Tomei Expressway and new expressways are shown in Figures 17 and 18. As the average Marshall stiffness is 30.6 in the Meishin Expressway, 31.2 in the Tomei Expressway, 34.8 in cold areas of new expressways and 37.2 in moderate areas, asphalt mixture is gradually becoming harder in expressways.

4.2.4/ Maximum Size

As an example of the result of the follow-up survey for test pavement of which maximum size is varied in cold areas, the results in the first and second stage of the Doo Expressway are shown in Figure 19 [12]. The following results were obtained.

- Wear resistance of mixtures with maximum size of 20 mm is superior to 13 mm. However, mixtures with maximum size of 40 mm are inferior in workability and durability to the 20 mm mixtures.
- A signficant difference of wear resistance is not observed between mixtures with or without rubber.

5/ MAINTENANCE AND REHABILITATION OF PAVEMENT IN COLD AREAS

5.1/ Evaluation of In-service Pavement and Target Maintenance Levels

Expressways are maintained and operated by an operation office which takes charge of a section of an expressway of 50 to 60 km in length and an operation bureau which supervises 7 to 10 operation offices.

The flow chart in Figure 20 shows the reasonable and systematic maintenance of pavement, or pavement management, being conducted on expressways [13]. The pavement characteristics were divided into two groups: a) roughness and bump, pot-hole, and localized cracking; and b) rutting and wear, cracking, and skid resistance. The former includes the pavement characteristics observed in limited areas and needs minor rehabilitation.

Hence, this is usually evaluated, judged and repaired by the individual operation office. The latter, on the other hand, is usually observed on a large scale and needs major rehabilitation. An operation bureau evaluates, judges and plans the repairs. Repair work itself is done by an appropriate operation office even in this case.

Pavement managment has three important phases: a) evaluation of pavement condition, b) establishment of target maintenance levels and comparison between maintenance levels and pavement conditions, and c) rehabilitiation work which includes planning, budgeting and repair work.

5.1.2/ Evalulation of Pavement Conditions

Evaluation of pavement condition is the first phase of pavement management. For minor rehabilitation characteristics, patrolling or visual inspection is the practical method because these characteristics are local and it is difficult to measure quantitatively and automatically. For major rehabilitation characteristics, however, innovational techniques suitable for the inventory study are adopted. The outline of measurement methods, used for evaluation of pavement conditions in expressways, is shown in Table 8 [14]. The method for measurement of rutting and cracking by high speed measuring vehicles is described in the following.

5.1.2.1/ Rutting

Rutting can be measured by the Rut Depth Photographic Recorder [15]. An illumination unit with a hair line projector projects the hairline on the rutted pavement. The projected hairline makes a shadow which is similar to, but an enlarged shape of, the actual shape of rutting. The vehicle's mounted pulse camera then takes the projected hairline vertically from 3.0 m in height. Since the measurement is taken by projecting the hairline, it is necessary to conduct this procedure at night.

The measuring principle is illustrated by Figure 21 and rut depth is calculated by Equation 2:

$$RD = \frac{H}{f} \times \frac{d_1 + d_2}{\cot \theta} \qquad (2)$$

where, RD = rut depth

H = photographing distance

$d_1 + d_2$ = photographed rut depth on the film

f = focal distance

θ = angle of hairline projection

Actually, $\theta = 26°33'$, H = 3 m and f = 15 mm, and substituting these figures for Equation 2, Equation 3 results:

$$RD = \frac{3{,}000}{15} \times \frac{d_1 + d_2}{2} = 100 \times (d_1 + d_2) \qquad (3)$$

Rut depth is defined as the vertical distance from the highest point to the lowest point within a lane as shown in Figure 22 [13].

Some operational features of rut depth measurement are summarized as below:

- measuring speed -- 0 - 80 km/h
- measuring width -- 4.6 m (max.)
- measuring interval — 5, 10, 20, 50 and 100 m
- measuring accuracy -- ± 2 mm

5.1.2.2/ Cracking

Cracking of the pavement can be continuously recorded by the Road Surface Photographic Recorder [15]. In determination of the cracked and patched areas, one-quarter lane width (= 0.875 m) x 1.0 mesh plate is overlapped on the film, and the number of the meshes covering the cracks and patchings is counted. The area of alligator cracking or patching is determined by the number of meshes counted multiplied by the individual mesh area ($0.875\ m^2$) and the area of longitudinal or transverse cracking is determined by the number of meshes multiplied by $0.875 \times 0.3 = 0.263\ m^2$. The crack ratio is calculated every 100 m by Equation 4:

$$\text{Crack ratio} = \frac{\text{cracking areas} + \text{patching areas}}{\text{measured areas}} \times 100 \qquad (4)$$

The photography is carried out at night since, in principle, there is less obstruction by traffic and adequate exposure under constant conditions can be obtained. Some operational features are summarized as below:

- measuring speed -- 10 - 60 km/h
- measuring width -- 5.0 m
- measurable minimum cracking — hair crack
- photographed scale -- 1:200

5.1.3/ Establishment of Target Maintenance Levels

The second phase of pavement management is the establishment of the target maintenance levels. After pavement conditions are measured, it is judged whether maintenance for rehabilitation is necessary. For judgment, it is necessary to set the target maintenance levels. The target maintenance levels for expressways are set as in Table 9 [13]. The figures in Table 9 are set for individual pavement characteristics. However, different types and degrees of distresses exist on actual road surfaces and, if it is necessary to decide the priority of rehabilitation on such a pavement, a synthetic evaluation is made by summing up each grade corresponding to the pavement value.

5.1.4/ Rehabilitation Work

The final phase of pavement managment is the execution of maintenance work in which planning, budgeting and examination of repair methods are included. In addition to technical examination for new pavement construction, the method of traffic control at the work site on the expressway, constraint of work time, and selection of construction equipment for the restricted work areas must be examined in detail.

5.2/ Present Pavement Conditions

Pavement conditions obtainable by high-speed measuring vehicles are rutting, cracking and skid resistance, at present, as described in Section 5.1. Measurements for rutting and cracking have been carried out on expressways since 1975. Total measured length as of March, 1982, is

34,000 lane-km (rutting 25,000 lane-km, cracking 9,000 lane-km). In this Section we shall describe the results of analysis of pavement conditions (rutting and cracking) in the last seven years.

5.2.1/ Rutting

Rutting, observed along the wheel path, is caused by various factors. The development of rutting varies with traffic characteristics and is classified into two types, namely, the wear type caused by studded tires and chained tires in cold areas, and the flow type caused by the passage of heavy wheel loads in moderate areas. Figure 23 shows a typical cross-section of rutting.

Figure 24 shows the first lane length with the rut depth more than 15 mm by year and areas. The graphs in Figure 24 show a rate of measurement defined by Equation 5:

$$\text{Rate of measurement} = \frac{\text{measured first lane length of each year}}{\text{first lane length in-service at the end of each year}} \qquad (5)$$

From Figure 24, it becomes clear that rutting of the wear type in cold areas shows a rapid rate of increase and rutting is shifting from the flow type to the wear type. This is attributable to the fact that the number of expressways in cold areas has increased since around 1972 and it takes about six or seven years for rut depth of the wear type to exceed 15 mm as in Figure 26.

Figure 25 shows the relative frequency distribution of rut depth in cut-and-fill, bridge, and tunnel sections. As in Figure 25, rut depth in the tunnel section is the least and in the cut-and-fill section is the most, except for the wear type. This may be caused by the effect of temperature: surface temperatures of pavement in tunnel and bridge sections are lower than in cut-and-fill sections during hot days in summer. In contrast, there is little difference in rut depth of the wear type in cut-and-fill, bridge, and tunnel sections. In particular, pavement in tunnel sections is worn uniformly and severely.

It is well-known that rutting progresses over the years and with increases in traffic. Figure 26 shows the yearly progress of rut depth. As in Figure 26, yearly progress of the wear type is three times faster than that of the flow type and a stronger relationship is observed between rut depth of wear type and years.

In order to ascertain the progress of rut depth caused by traffic, quantitatively, the progressive amount of rut depth per 10,000 passages of heavy commercial vehicles is calculated by Equation 6 and Equation 7:

$$\alpha = \frac{\sum_{i=1}^{n} \Delta \alpha_i}{n} \tag{6}$$

$$\Delta \alpha_i = \frac{RD_{i2} - RD_{i1}}{\Delta T_i} \tag{7}$$

where, α = average of coefficients of rut depth progress between two successive interchanges [mm per 10,000 passages of heavy commercial vehicles]

$\Delta\alpha i$ = coefficient of rut depth progress at i th point [mm per 10,000 passages of heavy commercial vehicles]

RD_{i2} = rut depth at a certain year at i th point [mm]

RD_{i1} = rut depth at the year before a certain year at i th point [mm]

ΔT_i = passages of heavy commercial vehicles in one direction during the rut depth increase from RD_{i1} to RD_{i2} at i th point [10,000 passages]

n = number of measured points between two successive interchanges

Calculated α values are plotted by area in Figure 27. As is evident from Figure 27, there is a considerable difference in α between cold areas and moderate areas. In cold areas, α are 0.37 - 0.05 (average 0.13) and in moderate areas are 0.24 - 0.008 (average 0.04). There is a dividing line between cold areas and moderate areas at α = 0.05 - 0.08.

Since the progress of rut depth in cold areas is 3.5 times faster than that in moderate areas and pavement in cold areas is worn uniformly, as mentioned above, it becomes clear that rutting in cold areas will become a more serious problem in future.

5.2.2/ Cracking

Present status of cracking, expressed by the first lane length with the crack ratio more than 10%, is shown in Figure 28. Graphs in Figure 28 use the rate of measurement in Equation 5 and show the following results.

- Only 150 lane-km out of 6,020 lane-km in-service show more than 10% crack ratio.
- Cracking is observed in the Meishin, Chuo, Chugoku, Kyushu and N. Tohoku Expressways and there is less difference in areas. However, it is considered that cracking in the N. Tohoku Expressway was mainly caused by pot-holes.

Cracking may generally occur and progress due to repeated traffic load, aging of asphalt cement, loss of subgrade and base course bearing capacity, reflection cracking, thermal effects in low temperatures, and so forth. Cracking tends to be observed when seven years (about 80 months) have passed or 2 to 3 million heavy commercial vehicles in one direction have passed after opening to traffic.

As mentioned above, the length of rutting needing rehabilitation is approximately 1,000 lane-km (600 lane-km of it in cold areas) and cracking is approximately 25 lane-km in expressways. Rutting, especially wear in cold areas, is a serious problem.

5.3/ Actual Situation of Maintenance and Rehabilitation

5.3.1/ Present Condition of Rehabilitation

Figure 31 shows the length of pavement rehabilitation in the last six years. Some conclusions follow.

- Approximately 300 lane-km are rehabilitated every year and there has been an increase in the last two years.
- The length rehabilitated in cold areas is short so far. However, it has been rapidly increasing since 1979 and this increases the total length of rehabilitation.
- Approximately 250 lane-km are rehabilitated every year in moderate areas and the Tomei and Meishin Expressways, which have been open to traffic over a long period of time and have heavy traffic, occupy the major portion of it.

Figure 32 shows the relationship between the rate of accumulated rehabilitation length calculated by Equation 8 and the accumulated heavy commercial vehicles:

Rate of accumulated rehabilitation length [%]

$$= \frac{\text{accumulated rehabilitation length between two successive interchanges}}{\text{first lane length between two successive interchanges}} \times 100 \qquad (8)$$

From Figure 32, the rate of rehabilitation tends to increase linearly as the heavy commercial vehicles increase and, roughly speaking, the greater part of the plots enters the range enclosed by solid lines. It is necessary to rehabilitate when the accumulated heavy commercial vehicles exceed 1 million vehicles, and the rate of rehabilitation becomes nearly 100% at 10 million vehicles. On the other hand, plots beyond the boundary in Figure 32 are gained from special areas where AADT is heavy and many vehicles using tire chains travel through in winter.

The number of years before the first rehabilitation is shown in Figure 33. In cold areas, rehabilitation has to be made once every four years, on average, and the frequency of rehabilitation in cold areas is about twice that in moderate areas.

5.3.2/ Rehabilitation Method

Overlay is usually conducted in expressways as a first rehabilitation alternative. When second or third rehabilitation is necessary, however, the rehabilitation method changes to overlay with surface milling because of shortage of guard cables or rails, vertical clearance problems at bridges, signs and tunnels, and vertical control problems with drainage facilities. The overlay with surface milling alternative has been conducted in full scale since around 1972 and has become a standard method of rehabilitation because of the improvement of a hot-milling machine. This method reaches completion from a technical viewpoint. However, there are some problems with the disposal of asphalt pavement debris as follows: a) it costs much to locate lawful dumping places or to dispose of this debris, and b) reusable asphalt mixtures are now scrapped. To solve these problems, and from the viewpoint of conservation of resources and energy, pavement recycling has been examined.

Pavement recycling is considered to have reached the stage for practical use in the world. In Japan, central-plant recycling is put to practical use at first [16]. Since expressways are designated to connect large cities, central-plant recycling is not economical. Furthermore, rehabilitation for rutting of the surface course occupies the greater part of pavement rehabilitation on expressways. Based on these circumstances, Nihon Doro Kodan has been studying surface recycling and has conducted some fundamental research since 1978. After obtaining the results of the fundamental research, and after the development of surface recycling equipment by private hands, trial recycling projects were done on expressways in 1981 [17]. These recycling projects are considered to be successful and follow-up surveys are being carried out to ascertain the durability of recycled mixtures.

6/ SUMMARY AND CONCLUSIONS

So far, the general situation of pavement on expressways in Japan, mainly in cold areas, has been outlined and the summary is as follows.

1/ Since the traffic and climatic conditions on expressways change from place to place, it is necessary to deal carefully with design, construction and maintenance of pavement.

2/ Pavement Design in Cold Areas

- If an expressway is situated in a frost area and if the frost penetration is greater than the total thickness of pavement, the materials of the subgrade within the depth of frost penetration should be replaced with materials unsusceptible to frost.
- Basically dense-graded asphalt mixtures are employed in cold areas in due consideration of wear resistance and skid resistance, and flow-resistance characteristics in summer. Mix design standards are set to gain a mixture that is combined hard coarse aggregates of large maximum size with rich asphalt mortar.
- As for the design of the road cross-section, in order to prevent the melted snow from flowing out to the roadway and freezing again, the median is designed in a depressed fashion and the shoulder is bent outwards.

3/ Maintenance and Rehabilitation of Pavement in Cold Areas

- Pavement management on expressways has three important phases: a) evaluation of pavement condition, b) establishment of target maintenance levels and comparison between maintenance levels and pavement condition, and c) execution of rehabilitation which includes planning, budgeting and repairing. In evaluation of pavement conditions, special vehicles with high travelling speed are used for measurement of rutting, cracking and skid resistance.
- Pavement rehabilitation on expressways is conducted to recover serviceability which has decreased, mostly by rutting. Pavement structural failure due to cracking is partially limited.
- As for pavement surface condition, rutting is a serious problem. In particular, progress of rutting of the wear type caused by studded

tires and chained tires in cold areas is 3.5 times faster than that in moderate areas.

- Frequency of pavement rehabilitation in cold areas is twice that in moderate areas.
- With extension of expressways into cold areas, it is predicted that the frequency of rehabilitation will increase rapidly and the expenditure of rehabilitation will become a serious problem. To cope with this problem, trial surface recycling projects have been carried out as an economical pavement rehabilitation alternative.

Pavement design, construction and maintenance on expressways are systematized in an outline based on the experience during the past 25 years. However, thinking of future construction and maintenance of several thousand kilometres of expressways which are to extend over the whole country, it is necessary to harmonize with the environment and to develop more economical technology. In particular, a great deal of effort is required for rehabilitation of abrasion caused by studded tires and chained tires in cold areas. Accordingly, in order to construct and maintain pavement economically, more reasonable and systematic design, construction and maintenance of pavement, such as design considering pavement life cycle and examination of target maintenance levels suitable to regional condition, are required.

To cope with these requirements, we must develop pavement management systems which make it possible to store pavement information, to utilize them in the following construction and maintenance of pavement, to forecast pavement performance and to design considering total costs during the design life. Furthermore, for the problems surrounding pavement including countermeasure of wearing, it is necessary to reconsider, not only from the viewpoint of pavement technology, but from broad standpoints such as traffic operation and legal restriction.

ACKNOWLEDGEMENTS

In preparing this paper, the authors are grateful to seniors in and out of Nihon Doro Kodan for permission to utilize their achievements.

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Table 1/ Test Conditions of Wear Test

Items	Condition
Test wheels	Studded snow tires (10.00-20, 14 PR)
Wheel load, Air pressure	2 ton, 6 kg/cm
Radius of circular track, Torque force	R = 3 m, Torque = ± 30 kgw m
Running speed	40 km/h
No. of wheel passes	0, 1.5, 3, 6, 10, 12.7 (x 10)
Surface condition	Wet
Temperature	0 °C
Amount of wheel shifting*	± 10 cm, ± 6 cm

* Amount of wheel shifting is ± 10 cm for the first half and ± 6 cm for the second half of every set up number of wheel passes.

Table 2/ Types and Gradations of Asphalt Mixes

Sieve sizes Types	Weight percent passing sieve							
	13	10	5	2.5	0.6	0.3	0.15	0.074
Dense graded asphalt mix	100	89.0	69.0	51.3	39.6	20.2	9.3	7.6
Wear resistant Topeka	100	93.5	78.0	61.1	47.4	24.5	11.3	9.3
Guss asphalt mix	100	92.6	74.9	55.1	44.6	35.2	28.4	26.1

Table 3/ Factors Contributing to Wear Resistance and Flow Resistance

Factors	Wear resistance	Flow resistance
Maximum size of coarse aggregate	The larger, the better	The larger, the better
Weight percent passing 2.38 mm sieve	The more, the better (Figure-6)	The less, the better (Figure-6)
Penetration grade of asphalt	The lighter, the better	The heavier, the better
Asphalt content	The more, the better (Figure-7)	The less, the better (Figure-7)
Quality of coarse aggregate	The harder, the better	———

Table 4/ Gradation of Asphalt Mixtures

Sieve sizes (mm)	Weight percent passing sieve(%)		
	Surface course		
	Type I	Type II	
	13mm	13mm	20mm
25			100
20	100	100	95~100
13	95~100	95~100	75~95
10	75~95	75~95	61~88
5	55~75	52~72	
2.5	38~58	40~60	
0.6	21~36	25~45	
0.3	13~25	16~33	
0.15	8~16	8~21	
0.074	4~8	5~12	

Note : Type I and II are applied in moderate areas and cold areas respectively

Table 5/ Classification of Asphalt Grade and Number of Blows to Determine the Optimum Asphalt Content

Traffic classification	Item	Regional classification		
		Warm	Moderate	Cold
Heavy	No. of blows	75	75	50
	Asphalt grade	60-80	80-100	80-100
Medium to light	No. of blows	50		
	Asphalt grade	80-100		

Notes 1) Warm ares :Areas with annual mean temperature of 14°C or more
Moderate areas :Areas with annual mean temperature less than 14°C and average year maximum snow of 30 cm or less
Cold areas :Areas of the annual maximum snow of 30 cm or more averaged in 10 years
2) Heavy traffic :Annual average daily traffic of heavy commercial vehicles in the first year after opening to traffic is 1,500 vehicles/day/one direction or more
Medium to light :Annual average daily traffic of heavy commercial vehicles in the first year after opening to traffic is less than 1,500 vehicles/day/one direction

Table 6/ Amount of Widening of Shoulder for Depositing Snow on Cut Section

Ten-year-period probable snow depth	0.5 m	1.0	1.5	2.0	3.0	3.5	more than 3.5
Amount of widening for depositing snow	0.0 m	0.5	1.0	2.0	3.0	4.0	5.0

Table 7/ Length of Expressways by Pavement Types and Areas (As of March, 1982)

Classification	Moderate areas	Cold areas	Total
Asphalt pavement	2044 67.9	838 27.8	2882 95.7
P.C.C. Pavement	116 3.9	12 0.4	128 4.3
Total	2160 71.8	850 28.2	3010 100

Upper : length (km), Lower : percentage (%)

Table 8/ Outline of Measurement Method for Evaluation of Pavement Conditions in Expressways

Measuring items	Measurement method or equipment	Required properties						Necessity of mechanization
		Continuity	High-speed	Traffic regulation	Easiness	Data processing	Investigation for cause of failure	
Rutting and abrasion	Rut Depth Photographic Recorder	suitable (every 10-100 m)	suitable (5-60 km/h)	not necessary	necessary to request for specific company	measured by trained analyzers	impossible	high
	Straight edge or leveling string	unsuitable	unsuitable	necessary	easy	manual	impossible	
Cracking	Road Surface Photographic Recorder	suitable (continuous photograph)	suitable (5-60 km/h)	not necessary	necessary to request for specific company	measured by trained analyzers	judgement from shapes of cracking is possible	high
	Sketch	unsuitable	unsuitable	necessary	easy	manual	"	
Skid resistance	Skid test vehicle	suitable (every 50-200 m)	suitable (60-80 km/h)	not necessary	driving skid test vehicle	digitalization using micro computer is possible	impossible	high
	British portable skid resistance tester	unsuitable	unsuitable	necessary	easy	manual	"	
Deflection	Benkelman beam	"	"	"	"	"	judgement of bearing capacity is possible	medium
	Dynaflect Road Rater	medium	medium (approx. 50 points/h)	"	acquisition of equipment is difficult now	"	"	
Roughness and bump	Profilometer (3 m or 8 m)	suitable	unsuitable (approx. 5km/h)	"	relatively easy	"	impossible	medium
	Leveling string or leveling	unsuitable	unsuitable	"	easy	"	"	
	Judgement by riding comfort during patrolling	"	suitable	not necessary	"	"	"	
Pot' hole	Visual inspection	"	unsuitable	"	"	"	judgement from shape and situation is possible	medium

Table-9 Target Maintenance Levels for Expressways and Arterial Toll Highways

Pavement Characteristics	Classification	Expressways		Toll Highways (Arterial)	
		Level A[*1]	Level B[*2]	Level A[*1]	Level B[*2]
Rut Depth (mm)	Through Lane	15	25	20	30
	Truck Climbing Lane	20	40	25	45
Crack Ratio (%)	All Site	20	40	20	40
Skid Number	Standard Site	30	25	30	25
	Important Site	35	30	35	30
Bump	Backfill of Abutment	-	20	-	30
	Backfill of Box Culvert	-	30	-	30
	Expansion Joint	-	15	-	20

[*1] Level A : Pre-Maintenance level in which immediate rehabilitation is not required but examination of distress causes and planning of rehabilitation should be considered.

[*2] Level B : Target level, or rehabilitation is desired to be done up to this level.

Photo-1 Traffic Simulator

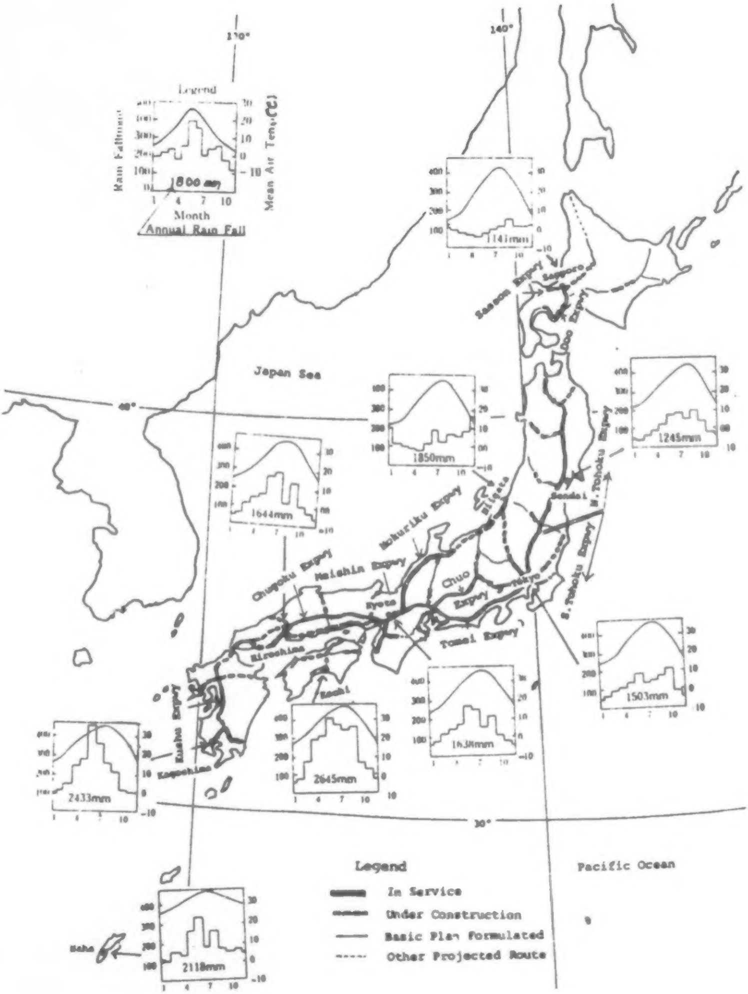
130°
140°
40°
30°
Legend
Rain Fall(mm)
Mean Air Temp.(℃)
Month
1800 mm
Annual Rain Fall
1141mm
1850mm
1644mm
1245mm
1503mm
1638mm
2645mm
2433mm
2118mm
Japan Sea
Pacific Ocean
Sapporo
Sasson Expwy
Dōō Expwy
Niigata
Sendai
N.Tohoku Expwy
S.Tohoku Expwy
Hokuriku Expwy
Chugoku Expwy
Maishin Expwy
Chuo Expwy
Tokyo
Kyoto
Tomei Expwy
Hiroshima
Kochi
Kyushu Expwy
Kagoshima
Naha
Legend
In Service
Under Construction
Basic Plan Formulated
Other Projected Route

Figure 1/ Network of Expressways and Climates in Japan
(As of March, 1982)

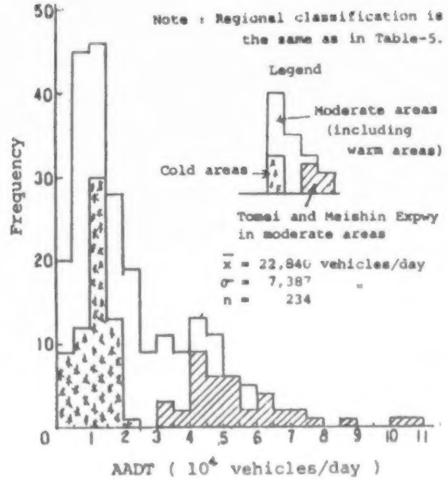

Figure 2/ Distribution of Annual Average Daily Traffic in 1980

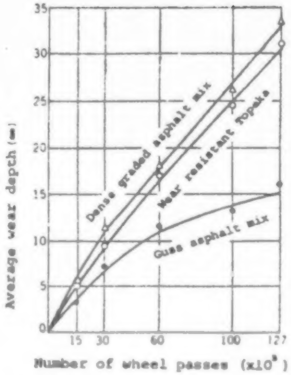

Figure 3/ Relationship between Wear and Types of Asphalt Mixes

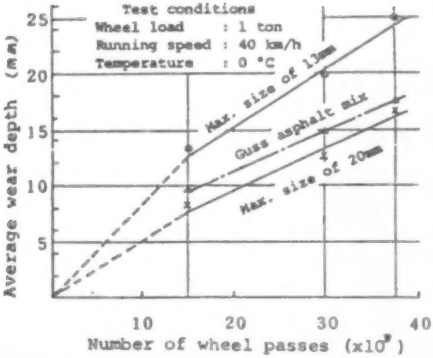

Figure 4/ Relationship between Maximum Size and Wear by a Chained Tire

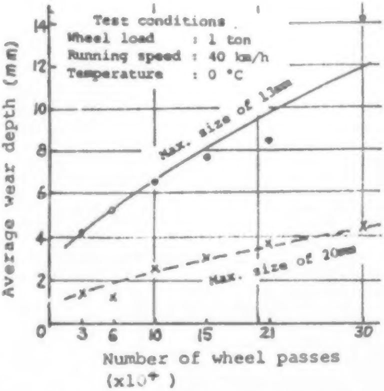

Figure 5/ Relationship between Maximum Size and Wear by a Studded Tire

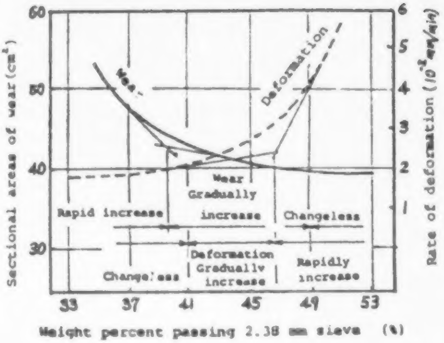

Figure 6/ Effects of Weight Percent Passing 2.38 mm Sieve

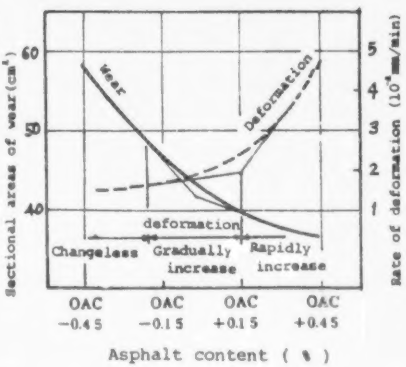

Figure 7/ Effects of Asphalt Content

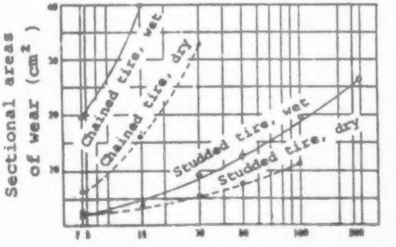

Figure 8/ Effects of Surface Conditions

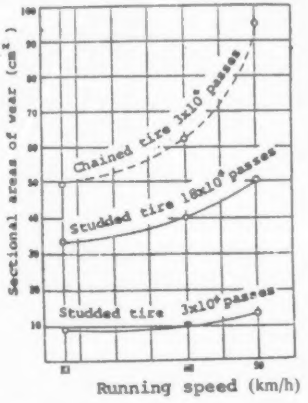

Figure 9/ Effects of Running Speed

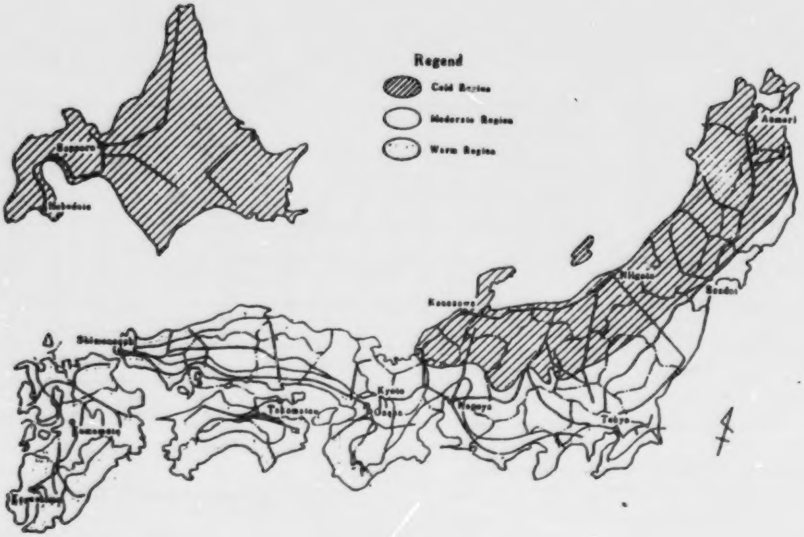
Regend
Cold Region
Moderate Region
Warm Region
Sapporo
Hakodate
Aomori
Niigata
Sendai
Kanazawa
Shimonoseki
Kyoto
Osaka
Nagoya
Tokyo
Takamatsu
Kumamoto
Kagoshima

Figure 10/ Regional Classification for Selecting Asphalt Penetration Grade and Number of Blows for Marshall Test

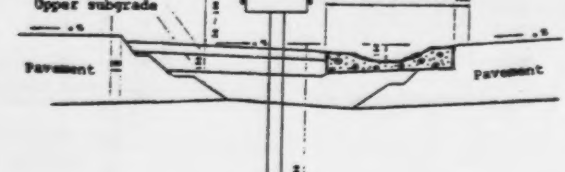
Margin strip
Median strip
Margin strip
Guard cable
Cement concrete
Rolled-gutter
Upper subgrade
Pavement
Pavement

Figure 11/ Example of Median Strip in Cold Areas

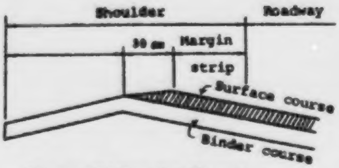
Shoulder
Roadway
30 cm
Margin
strip
Surface course
Binder course

Figure 12/ Bending Shoulder

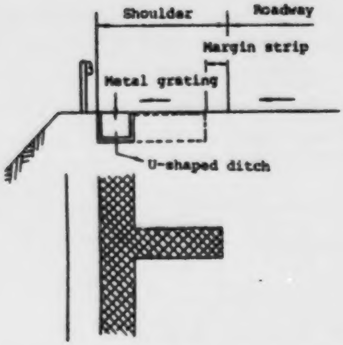
Shoulder
Roadway
Margin strip
Metal grating
U-shaped ditch

Figure 13/ Example of Shoulder Drainage Facilities

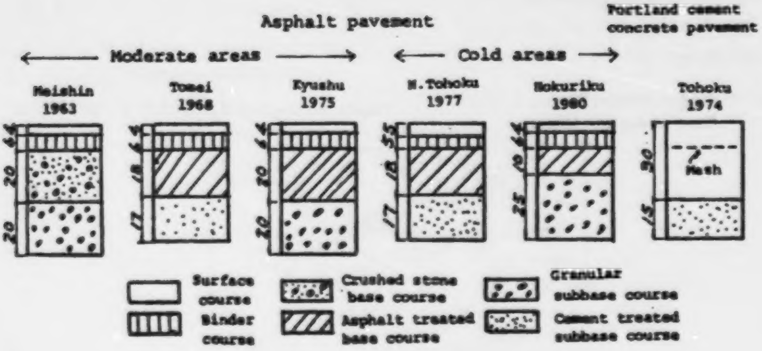
Asphalt pavement
Portland cement concrete pavement
Moderate areas
Cold areas
Meishin 1963
Tomei 1968
Kyushu 1975
N.Tohoku 1977
Hokuriku 1980
Tohoku 1974
4
4
20
20
5
6
18
17
4
4
20
20
5
5
18
17
4
4
10
25
30
15
Mesh
Surface course
Binder course
Crushed stone base course
Asphalt treated base course
Granular subbase course
Cement treated subbase course

Figure 14/ Typical Pavement Cross-Section

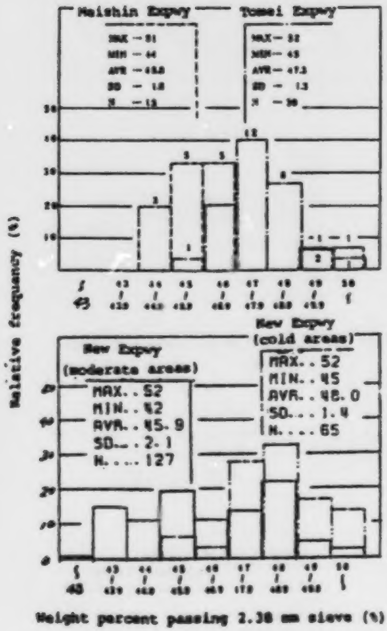

Figure 15/ Weight Percent Passing 2.36 mm Sieve of Surface Mixes

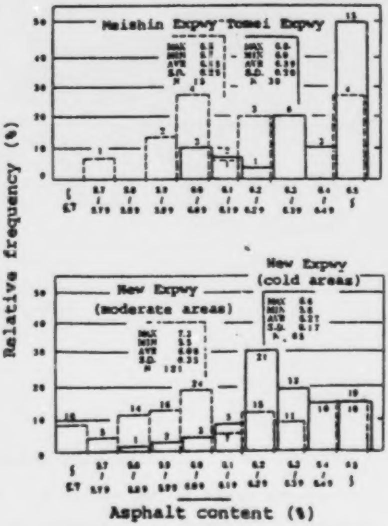

Figure 16/ Asphalt Content of Surface Mixes

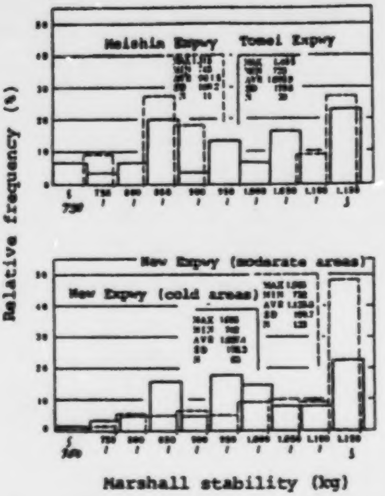

Figure 17/ Marshall Stability of Surface Mixes

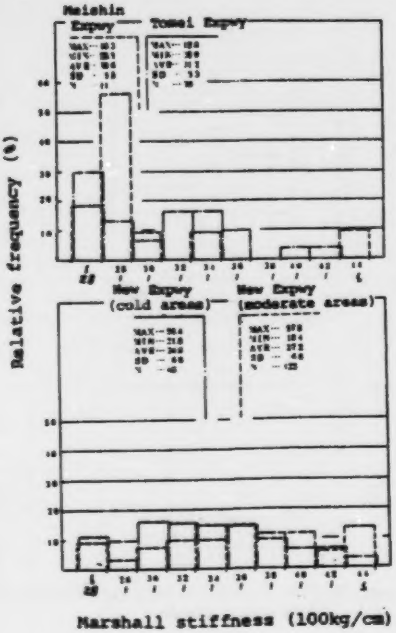

Figure 18/ Marshall Stiffness (Stability/Flow) of Surface Mixes

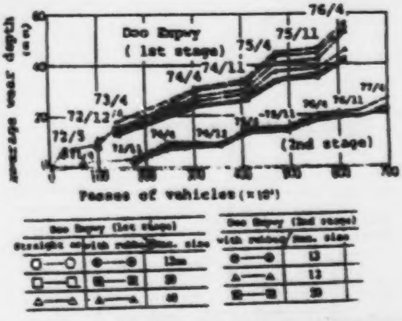

Figure 19/ Example of the Result of Follow-up Survey

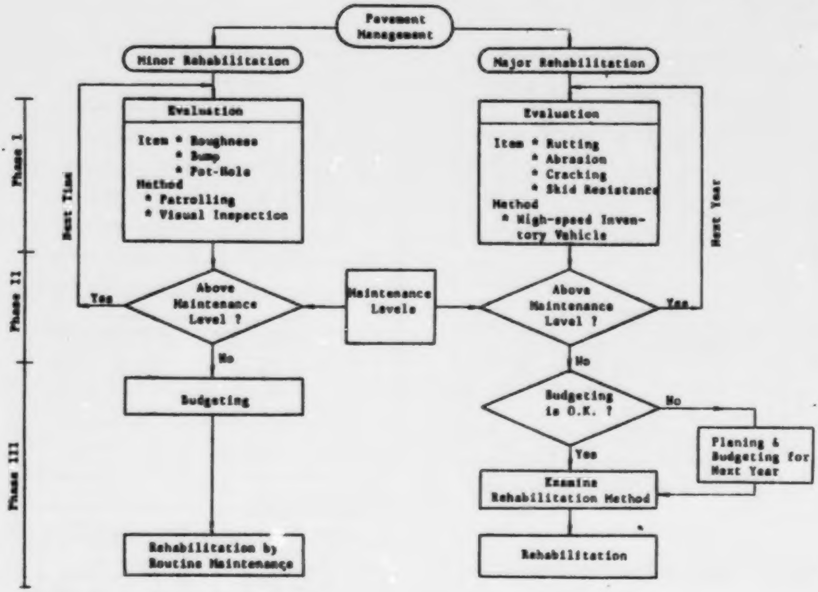
Pavement Management
Minor Rehabilitation
Major Rehabilitation
Phase I
Phase II
Phase III
Next Time
Next Year
Evaluation
Item * Roughness
* Bump
* Pot-Hole
Method
* Patrolling
* Visual Inspection
Evaluation
Item * Rutting
* Abrasion
* Cracking
* Skid Resistance
Method
* High-speed Inventory Vehicle
Yes
Above Maintenance Level ?
Maintenance Levels
Above Maintenance Level ?
Yes
No
No
Budgeting
Budgeting is O.K. ?
No
Yes
Planing & Budgeting for Next Year
Examine Rehabilitation Method
Rehabilitation by Routine Maintenance
Rehabilitation

Figure 20/ Flow Diagram of Pavement Management Applied for Expressway System

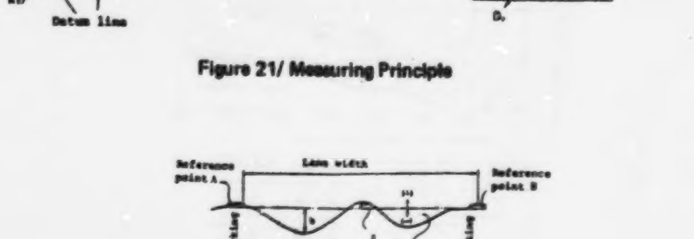
Pulse camera
Hair line projector
RD
Datum line
Focal distance f
Photographing distance
Hair line projector
Datum line
RD
Rut depth

Figure 21/ Measuring Principle

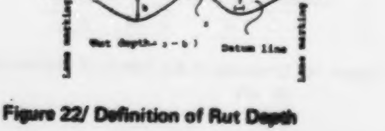
Reference point A
Lane width
Reference point B
b
a
Rut depth= (a - b)
Datum line
Lane marking
Lane marking

Figure 22/ Definition of Rut Depth

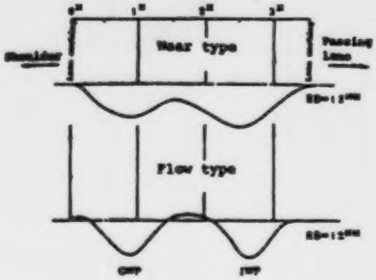
0m
1m
2m
3m
Shoulder
Wear type
Passing lane
Flow type
OWP
IWP

Figure 23/ Typical Cross-Section of Rutting

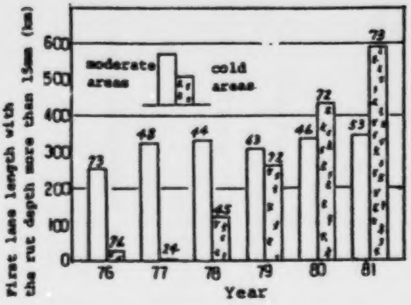

Figure 24/ Lane Length with the Rut Depth More than 15 mm

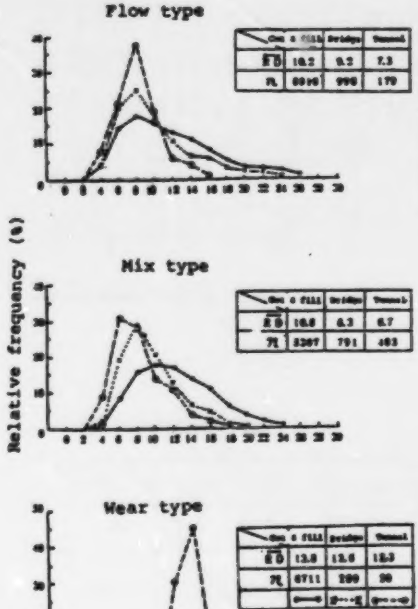

Figure 25/ Rutting in Cut and Fill, Bridge and Tunnel Sections

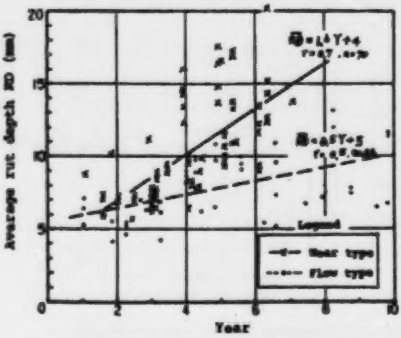

Figure 26/ Yearly Progress of Rut Depth

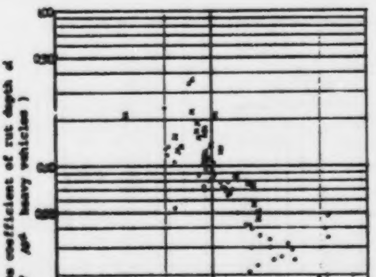

Figure 27/ Progress Coefficient of Rut Depth

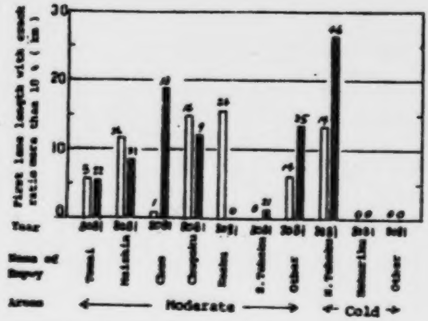

Figure 28/ Lane Length with the Crack Ratio More than 10%

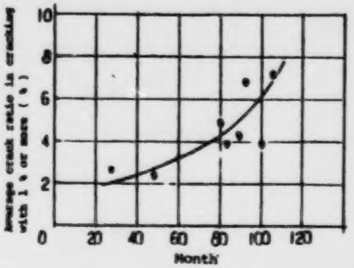

Figure 29/ Yearly Progress of Cracking

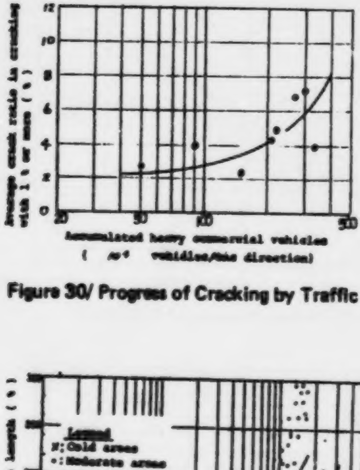

Figure 30/ Progress of Cracking by Traffic

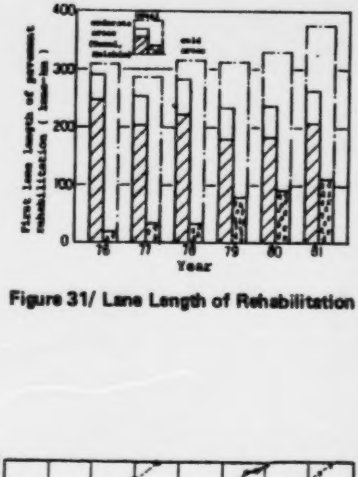

Figure 31/ Lane Length of Rehabilitation

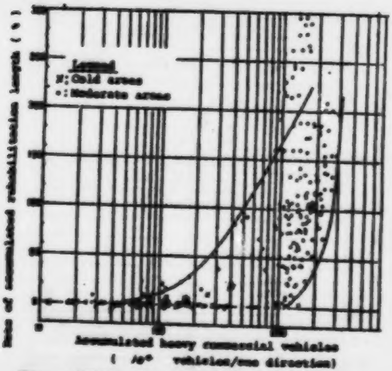

Figure 32/ Pavement Rehabilitation and Heavy Commercial Vehicles

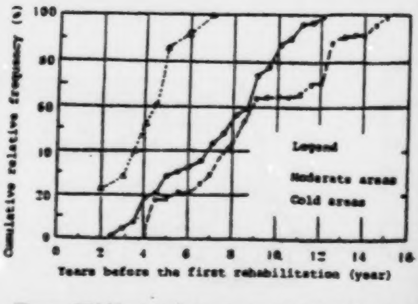

Figure 33/ Years before the First Rehabilitation

Session 2

The Effects of Freeze-Thaw on the Deformation Properties of Subgrade Soils for Highway and Airport Runways in Cold Regions

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The Effects of Freeze-Thaw on the Deformation Properties of Subgrade Soils for Highway and Airport Runways in Cold Regions

by

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Abstract

Climatic conditions of the north central portion of North America are largely responsible for the seasonal variations that occur in the deflection of pavement structures when they are subjected to heavy wheel loads Typically, deflections are small during the winter months when the subgrade is frozen. Moderation of the weather during the months of April and May causes thawing of the base, subbase and subgrade material, which in turn results in increased deflections, high stresses and reduced pavement life. As the summer season progresses, the subgrade materials gradually regain their stiffness, thereby reducing deflections and improving the performance of the pavement structure.

The changes in subgrade stiffness, as quantified by the resilient modulus, are due largely to changes in moisture content, to moisture movement, to the development of a secondary soil structure, and to the resulting changes in soil matric suction.

Various studies at the University of Saskatchewan and the Saskatchewan Department of Highways have provided a substantial amount of data that clearly indicate the role of freeze-thaw cycles in producing this moisture movement and the concomitant change in soil properties. It is apparent that two or three cycles of freeze-thaw cause the formation of a pronounced secondary soil structure that appears to subsequently control the behavior of these fine grained subgrade soils. Although the global water content of the soil may remain essentially unaltered, moisture movement within the soil mass is responsible for decreases in matric suction and the resulting

decreases in resilient modulus. These results are illustrated and the data is reviewed in this paper. In addition, the variables controlling the behavior of partially saturated fine grained soils are described. Some recent methods for measuring soil matric suction in the laboratory and field are considered, so that the input parameters to the constitutive equations can be more adequately quantified. Finally some statements are made concerning the maturity of these aspects of subgrade design, and recommendations are provided for further experimentation.

Introduction

The design of highway and airport runway pavement structures in the north central portion of North America is profoundly influenced by the climatic conditions of these regions. Not only does the design of the comparatively thin asphalt concrete layer overlying the base and subbase materials require special attention because of cold temperature cracking, but the behavior of

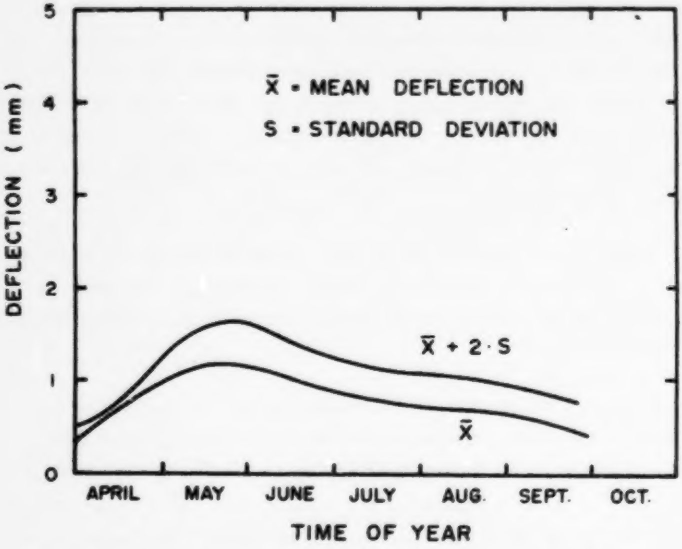

Figure 1/ Benkelman Beam Deflection (Bergan and Monismith, 1973) the subgrade material due to frost action is of major significance as well.

Results of Benkelman beam tests indicate large seasonal variations in the magnitude of the deflections (e.g. Figure 1). Typically, deflections are small throughout the winter months, increasing to a maximum in May and June, and then decreasing during the summer and fall months. Since the results of Benkelman beam tests may be related, at least in a qualitative sense, to the dynamic or resilient response characteristics of the subgrade materials (Bergan 1972), it becomes increasingly important to assess the effects that seasonal variations in climate have on the mechanical properties of these materials.

A number of studies have been conducted at the University of Saskatchewan, by the Saskatchewan Department of Highways, and other agencies in an attempt to identify and quantify the parameters or factors which apparently affect the stiffness or resilient modulus of the subgrade materials, e.g., Sauer and Monismith 1968, Mickleborough 1970, Culley 1971, McLeod 1971, Bergan 1972, Weimer 1972, Fredlund et al 1977, and others.

The primary purpose of this paper is to review some of the research, especially from Saskatchewan, regarding the effect of frost action on moisture movements, moisture content, the development of a secondary soil structure, the changes in matric suction and the resulting changes in resilient modulus.

Some methods of measuring soil matric suction are described in the latter part of the paper and brief comments are made regarding their use and reliability.

Finally, some statements are made concerning the maturity of these aspects of subgrade design, and recommendations are provided for further experimentation.

Climate

Although the introduction referred to a rather extensive region (i.e., north central North America) where climatic conditions may be somewhat similar, there will in fact be large variations in temperatures and precipitation. Since much of the experimental work that is referred to in this paper was collected in a rather small region of the general area, it is of interest to briefly describe the nature of the climate of Saskatchewan (Figure 2).

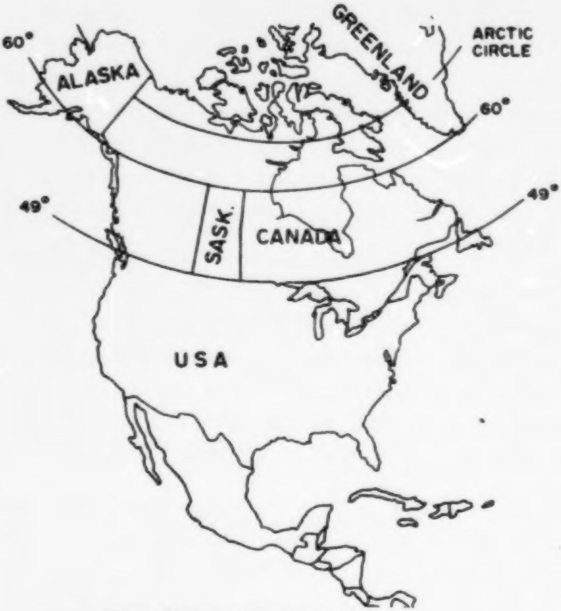

Figure 2/ Map of North America

Although Saskatchewan extends from 49° 00' to 60° 00' N latitude and from 101° 22' to 110° 00 W longitude most of the primary road systems extend from southern boundary to 55° N. Even within this region the climate varies from semi arid in the southwest to subhumid in the northeast. The climate is essentially continental in nature with long cold winters and short warm to hot summers. The mean annual air temperatures vary from 1°C to 2°C from the north to south regions. The normal freezing index is 3000 to 4000 C° days and the normal thawing index is 4000 to 5000 C° days.

There is approximately 41 cm of annual precipitation of which 30% to 40% consists of snowfall (Richards and Fung 1969).

Soil Characteristics

Essentially all of the road systems in Saskatchewan are located in two physiographic areas referred to as the Saskatchewan Plains Region of the

Central Lowlands Province and the Great Plains Province. The elevation of these regions vary from about 180 to 730 m (600 to 2400 ft.) with the majority between 365 and 550 m (1200 to 1800 ft.) msl. Since almost all of this area has been glaciated the region is covered by ground moraine, lacustrine deposits, outwash, and other lesser landforms. However, the two materials that have been tested most extensively, mainly because of their predominance, are the proglacial lacustrine lake clays and the clay tills. Although the properties of these two materials will vary throughout the region, Table 1 provides typical values for the materials that have undergone extensive testing.

Table 1

Soil Properties

Property	Glacial Till	Lacustrine Clay (Regina Clay)
Liquid Limit	38%	78%
Plastic Limit	17%	30%
Sand	27%	5%
Silt	40%	27%
Clay	33%	68%
Specific Gravity	2.77	2.83
Max Std Density	1.70 Mg/m^3(106 pcf)	1.47 Mg/m^3(92 pcf)
Opt. Water Content	19%	28%

After Fredlund 1975.

Laboratory Equipment

Loading Mechanisms

Repeated loading equipment modelled along the lines of Seed and Fead (1959) was developed by the Saskatchewan Department of Highways Laboratories in 1963. The samples were mounted in an enclosed plastic chamber and the loads were applied by a rotating cam-loading mechanism in conjunction with a hangar loading frame (Sauer and Monismith 1968). The first repeated loading machine was developed at the University of Saskatchewan in 1968. It has been revised and updated numerous times over the ensuing period. The present version is shown in Figure 3.

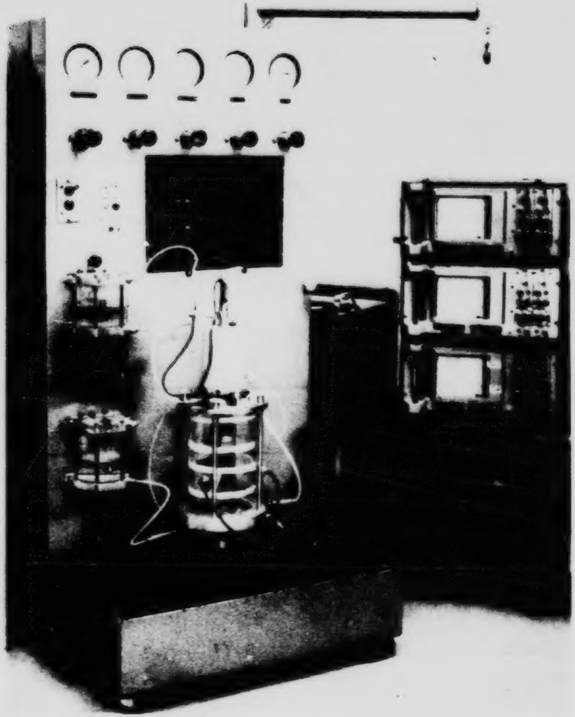

Figure 3/ Repeated Loading Triaxial System
(University of Saskatchewan, Saskatoon)
(S. Mizinski Photo)

The sample of soil encased in a rubber membrane is placed in a reinforced triaxial chamber. A static confining pressure is applied to the specimen by compressed air. The vertical stress is applied as a pneumatic pressure to a bellofram located above the main piston. A typical loading frequency is 20 repetitions per minute with a load duration of 0.1 sec.

Deformations

The vertical load is measured by a load cell in the base plate and the vertical and lateral strains are measured with linear variable differential transducers (LVDT).

Pore Pressures

Attempts to measure pore water pressure when the sample is being subjected to dynamic loads have been largely unsuccessful. All devices employing the use of high air entry stones placed at the base of the sample have not proved to be satisfactory because of the slow response of the measuring system to changes in water pressure (Fredlund and Morgenstern 1973).

Morrison (1980) installed temperature-compensated thermocouple psychrometers in the loading cap, at the third points, and at the mid height of the specimens. The psychrometers provided a measure of total suction (matric + solute) to an accuracy of ± 41 kPa (5.9 psi). Although psychrometers measure total suction it was reasoned that the solute component should remain constant during the loading cycle and that changes in total suction should be due to changes in the matric component only.

Morrison (1980) does not recommend the psychrometric method for general soil testing as it requires very sensitive equipment that is costly and prone to damage. It is only suitable under controlled conditions that exist only in highly sophisticated research laboratories.

Since methods to monitor suction under dynamic loads have not been well established, the general procedure is to measure the suction on a pressure plate apparatus after the test has been completed.

Recent work by Cole et al (1981) and Chamberlain and Cole (1982) indicate that matric suction may be monitored during dynamic loading by using porous ceramic tensiometer tips that are inserted into the bottom of the specimen. However, these devices are only capable of monitoring suction values of less than 1 atmosphere pressure (100 kPa).

Resilient Modulus

The resilient modulus is a measure of the stiffness of the untreated materials comprising the pavement structure.

$$M_R = \frac{\sigma_1 - \sigma_3}{\epsilon_1} \qquad [1] \text{ Seed et al 1955}$$

where

M_R = modulus of resilient deformation

σ_1 = major principal stress

σ_3 = minor principal stress

ε_1 = recoverable or resilient strain the direction of the major principal stress.

Other variations of [1] using the stress invariants, the octahedral shear stress, and suitable constants may provide a more complete description of the soil behavior (e.g. Cole et al 1981). It would seem, however, that matric suction should be included as an independent variable and that the effect of density should appear as well.

Research efforts across North America on subgrade soils subjected to repeated loading by Seed et al (1955), Seed and McNeil (1956), Larew and Leonards (1962), Seed et al (1965), Sauer and Monismith (1968), Dehlen (1969), Culley (1971), Bergan (1972), Weimer (1972), Fredlund et al (1977), Johnson et al (1978), Cole et al (1981), to name but a few have allowed the following general conclusions to be drawn.

1. Resilient modulus decreases with increases in dry density.
2. Resilient modulus decreases with increases in water content especially dry of optimum water content.
3. Resilient modulus increases with increases in the number of stress applications as long as the stress level is below failure.
4. Resilient modulus generally decreases with increases in deviator stress especially at low deviator stresses.
5. Resilient modulus increases with increases in confining pressure. Seed et al (1965) found the relationship of resilient modulus vs confining pressure to be linear on a log-log plot for sand while Fredlund et al (1977) found it to be linear on an arithmetic plot for a sandy, silty, clay till.
6. Resilient modulus increases with increases in soil suction.
7. Resilient modulus decreases after one or more cycles of freeze thaw.

The effect of density, moisture content, confining pressure, deviator stress, and number of cycles of loading on resilient modulus as noted in the first five observations outlined above are well documented in the cited references and do not require further review here.

Matric Suction

In view of the more recent understanding of the behavior of unsaturated soils (Fredlund and Morgenstern 1977; Fredlund 1979) the role of matric suction and the effect of changes in matric suction on resilient modulus are of interest.

Much of the original work regarding the effect of soil matric suction on subgrade soils was performed by Croney et al (1958) in England, by Aitchison and Richards (1965) in Australia and by de Bruijn (1965) and Blight (1965) in South Africa. In North America Sauer and Monismith (1968) described a field and laboratory investigation that indicated the influence of matric suction on the resilient modulus of a typical glacial clay till from Southern Saskatchewan. Later studies by Mickleborough (1970), McLeod (1971), Culley (1971), Bergan (1972), Fredlund (1977), Cole et al (1981), Johnson et al (1982) and others provided additional data to show the effect of matric suction on resilient modulus and the qualitative relationship be-

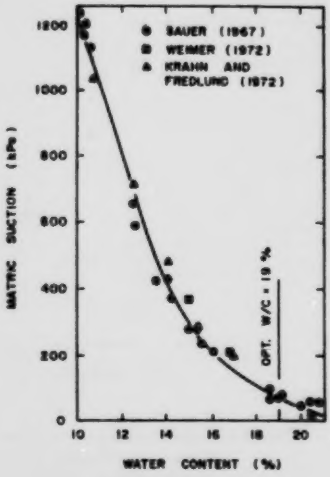

Figure 4/ Water Content vs Matric Suction for Glacial Till (Fredlund et al, 1975)

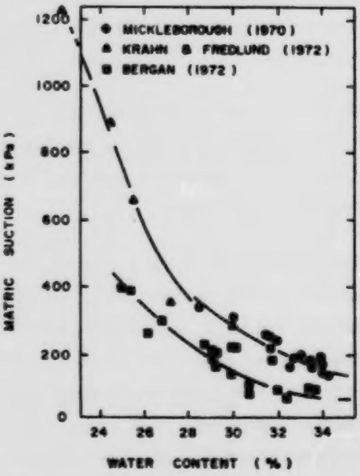

Figure 5/ Water Content vs Matric Suction for Regina Clay (Fredlund et al, 1975)

tween frost action-matric suction-resilient modulus.

In most cases the relationship between matric suction and resilient modulus is established by first determining the relationship between water content and matric suction and then between water content and resilient modulus. A cross plot of the resulting diagrams provides the relationship between matric suction and resilient modulus. Figure 4 and Figure 5 show typical relationships between matric suction and water content for the two soils described earlier.

Figure 6 and Figure 7 show typical relationships between resilient modulus and water content for the same soils. Cross plotting Figure 4 and Figure 6 provides Figure 8, i.e. resilient modulus vs matric suction for glacial till. Similarily Figure 9 is generated from Figures 5 and 7.

Frost Activity

When the samples of soil are subjected to freeze-thaw in a closed system (i.e. constant water content) they undergo a remarkable decrease in matric suction, e.g. Figure 10 and Figure 11.

Repeated load tests performed on these samples indicate a substantial decrease in resilient modulus, e.g. Figure 12 and Figure 13.

The decrease in matric suction is caused by redistribution of moisture within the soil mass during the freezing process. Moisture in the form of liquid and vapor move under the influence of temperature induced gradients. This moisture collects and forms minute ice lenses throughout the soil mass. In addition a pronounced secondary or nuggety structure is developed in the soil after two or

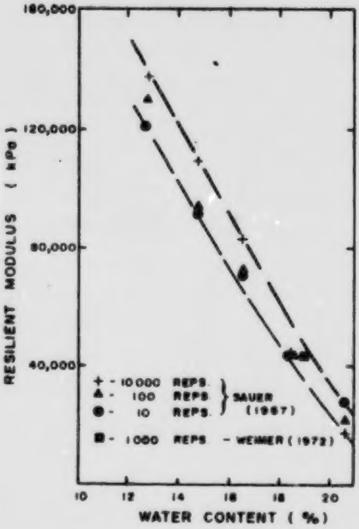

Figure 6/ Water Content vs Resilient Modulus for Glacial Till (Fredlund et al, 1975) three cycles of freeze thaw. Ice crystals are clearly evident on these fracture planes when the soil is frozen. When the soil thaws the water remains on the surface of the nuggets and the behavior of the soil mass is then controlled largely by these films of water.

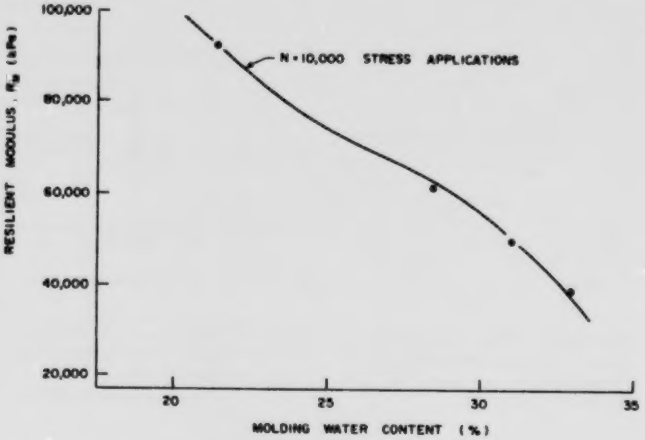

Figure 7/ Matric Suction vs Resilient Modulus for Regina Clay (Mickelborough, 1970)

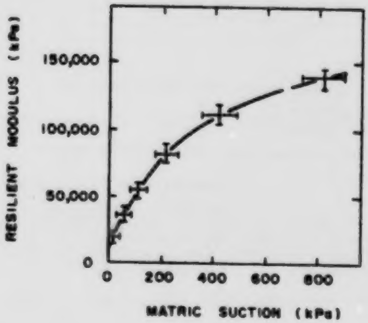

Figure 8/ Matric Suction vs Resilient Modulus for Glacial Till (Fredlund, 1975)

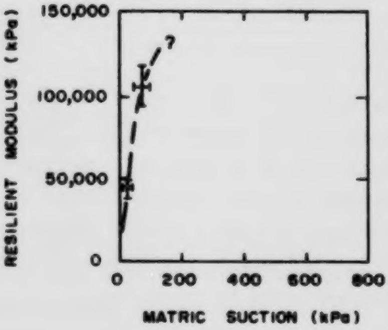

Figure 9/ Water Content vs Resilient Modulus for Regina Clay (Mickelborough, 1970)

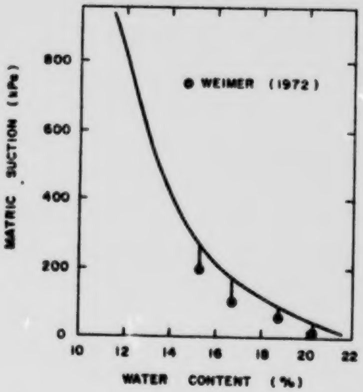

Figure 10/ Reduction in Matric Suction for Glacial Till Subjected to Freeze-Thaw Cycles (Weimer, 1972)

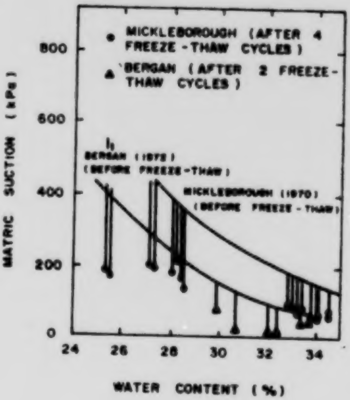

Figure 11/ Reduction in Matric Suction for Regina Clay Subjected to Freeze-Thaw Cycles (Fredlund et al,1975)

Eventually as the moisture redistributes itself under reversed moisture gradients the material tends to regain its original stiffness. This redistribution process is greatly enhanced by subjecting the soil to numerous load repetitions (Figure 12).

There does not appear to be any preferred orientation to the fracture planes or ice lenses that occur within the soil mass as might be anticipated in view of the obvious direction of the temperature gradient. However mapping of these fracture patterns with the aid of a microscope might provide more insight into this phenomenon. Figures 12 and 13 are examples of the decrease in resilient modulus that is observed for the same soils when subjected to freeze-thaw.

Obviously the stress state variables controlling the soil behavior have been substantially altered by the freeze-thaw cycles.

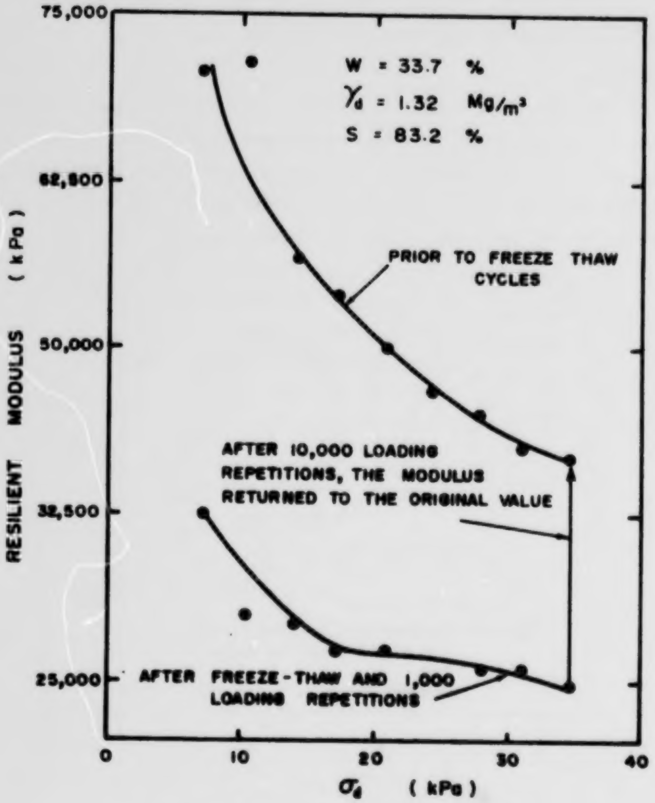

Figure 12/ Resilient Modulus Tests Before and After Freezing for Regina Clay (Fell, 1970) (Bergan and Fredlund, 1973)

Volume Changes During Freeze-Thaw

Even in closed systems that do not allow any general influx or exit of water, small changes in volume generally occur during the freeze-thaw cycle. The amount and direction of volume change is dependent upon the density and degree of saturation for these cohesive soils. Hamilton (1966) found that samples shrank or decreased in volume when frozen for degrees of saturation less than 90 percent while those compacted at a degree of saturation greater than 90 percent increased in volume. Generally, a net increase in volume occurred after thawing. Similar observations have been made by Lidgren 1970 and Culley 1970 (Figure 14).

Swelling Pressure

A measure of the effective stress within the soil skeleton may be determined by the one dimensional constant volume swelling pressure test. These tests performed on samples of cohesive soil before and after cycles of freeze-thaw indicate that a substantial decrease in the swelling pressure occurs after two or three cycles of freeze thaw.

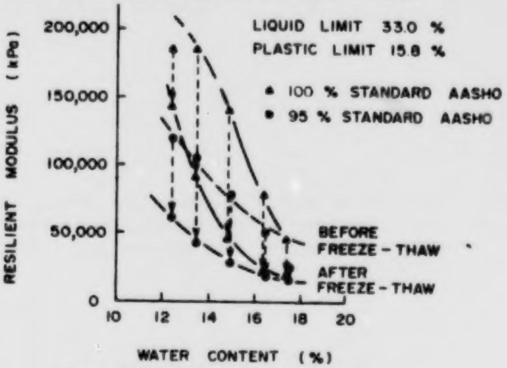

Figure 13/ Resilient Modulus Tests Before and After Freezing for Glacial Till (Culley, 1971)

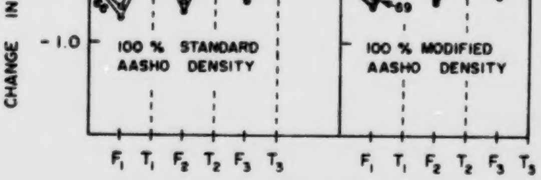

Figure 14/ Effect of Freeze-Thaw on Volume Change for Glacial Till (Culley, 1971)

Figure 15 shows the decrease in swelling pressure after three cycles of freeze-thaw. The results indicate that a larger decrease occurs at high densities and at water contents dry of optimum.

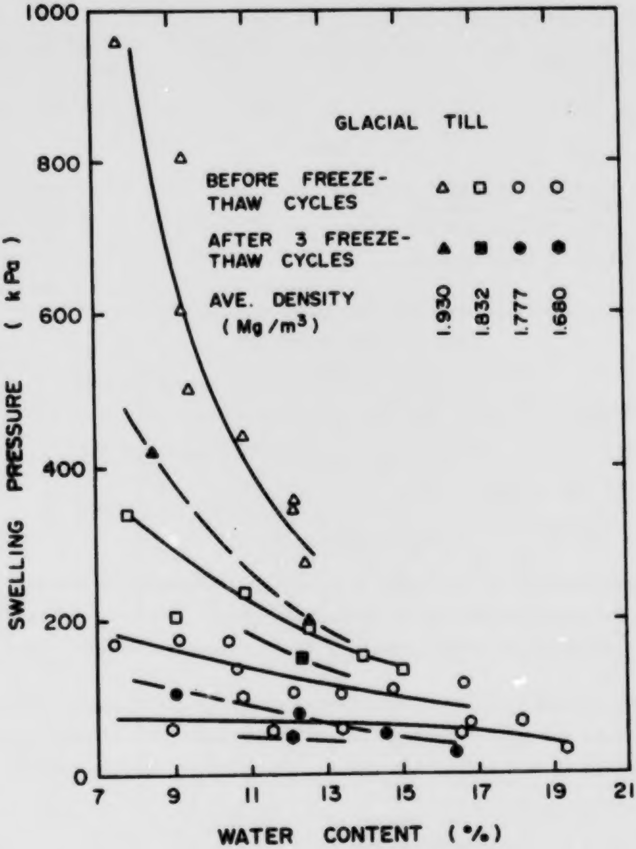

Figure 15/ Water Content vs Swelling Pressure Before and After Freeze-Thaw for Glacial Till (After Lidgren, 1970)

Unsaturated Soil Behavior

If a soil is dry or saturated equation [1] may adequately describe the relationship between deviator stress and resilient strain. However, if the soil is unsaturated the equation required to describe the resilient modulus is more complex. Fredlund and Morgenstern (1977) and Fredlund (1979) have shown that in general at least two of the three stress state variables $(\sigma - u_a)$, $(\sigma - u_w)$, $(u_a - u_w)$ are required to predict the behavior of unsaturated soils.

where

σ = total applied stress
u_a = pore air pressure
u_w = pore water pressure
$(u_a - u_w)$ = matric soil suction

It can then be reasoned that the resilient modulus (M_R) is:

$$M_R = \text{fn}\ [(\sigma_1 - u_a), (\sigma_3 - u_a)(u_a - u_w)] \quad . \qquad [2]$$

(Fredlund et al 1975 and Fredlund et al 1977)

Under conditions of saturation $u_a = u_w$ and [2] reverts to [1]. Equation [2] can be recast as:

$$M_R = \text{fn}\ [(\sigma_3 - u_a), (\sigma_1 - \sigma_3), (u_a - u_w)] \qquad [3]$$

Ideally, verification of [3] would require the monitoring of air and water pressures during the application of dynamic loads. Serious technical problems have hampered the measurement of these variables as noted previously.

Fredlund et al (1977) related resilient modulus to confining pressure, deviator stress, and matric suction by [3]. The magnitude of matric suction was determined at the end of the tests. Equation [3] was formulated from Figure 16 as:

$$\log M_R = c_{1d} - m_{1d}\ (\sigma_1 - \sigma_3) \qquad [4]$$

where

c_{1d} = the intercept on the ordinate

m_{1d} = the slope of the line on the semi log plot

Since c_{1d} and m_{1d} are functions of matric suction, the deviator stress $(\sigma_1 - \sigma_3)$ is also related to matric suction.

A series of repeated load tests performed in that study on a glacial till (similar to that described earlier) with different densities and

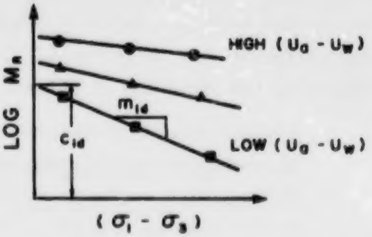

Figure 16/ Deviator Stress vs Log of Resilient Modulus for Glacial Till (Fredlund et al,1977)

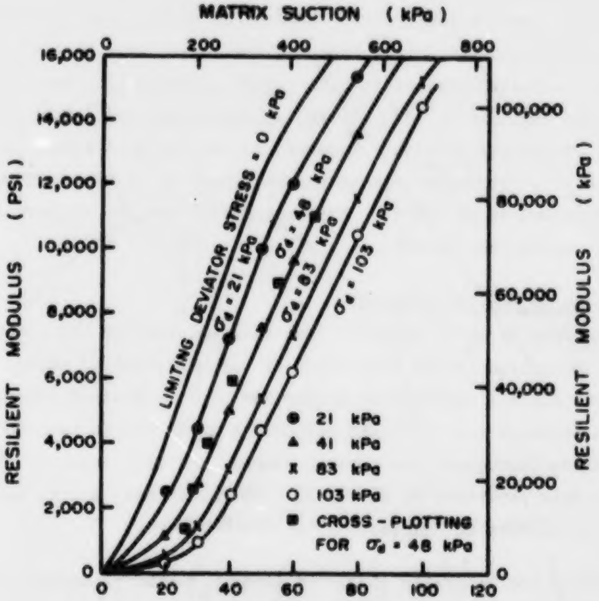

Figure 17/ Matric Suction vs Resilient Modulus for Glacial Till (Fredlund et al,1977) matric suctions indicated that [4] may be used to describe the relationship between resilient modulus, deviator stress, confining pressure and matric suction. It was also apparent that confining pressure had a negligible effect on the results. Figure 17 shows the results of resilient modulus vs matric suction for different dry densities. Recommendations for establishing the variables in [4] are to:

1. Compact samples at various water contents and densities with a compactive energy comparable to that used in the field.
2. Subject each sample to at least 100 repititions of deviator stress ranging from 20 to 80 kPa.
3. Determine the matric suction from two or three smaller samples obtained from the test specimen after the test is completed.

Problems with reproducibility indicate that three specimens should be prepared and tested for each density and water content.

Morrison (1980) used a temperature-compensated thermocouple psychrometer to measure total suction during repeated load tests on a glacial till material as indicated earlier in this report. The results were used to establish the validity of [4]. It was apparent that the intercept c_{1d} was a function of initial suction. However, it was not well established that the slope m_{1d} was strongly related to the values of initial suction as shown by Fredlund et al (1977). Morrison (1980) did not subject any soil samples to freeze-thaw cycles.

Field Methods to Monitor Suction

For a given density it is apparent that soil moisture or the state of the energy in the soil moisture quantified by the magnitude of matric suction largely controls the deformation properties of fine grained subgrade soils. It is also apparent from pavement deflection measurements that the matric suction varies throughout the spring, summer, and fall seasons. Efforts to monitor this parameter in the field throughout these months have met with varying degrees of success.

Subgrade soils are generally placed at maximum AATSHO (Standard Proctor) densities and corresponding optimum moisture contents. Under these conditions of placement, matric suctions are in the order of 150 to 350 kPa. Present knowledge indicates that after a period of one or two years the soil reaches an equilibrium moisture content with the environment which is well in excess of optimum (e.g. Figure 18). Decreases in matric suction accompany this increase in water content and typical matric suctions are

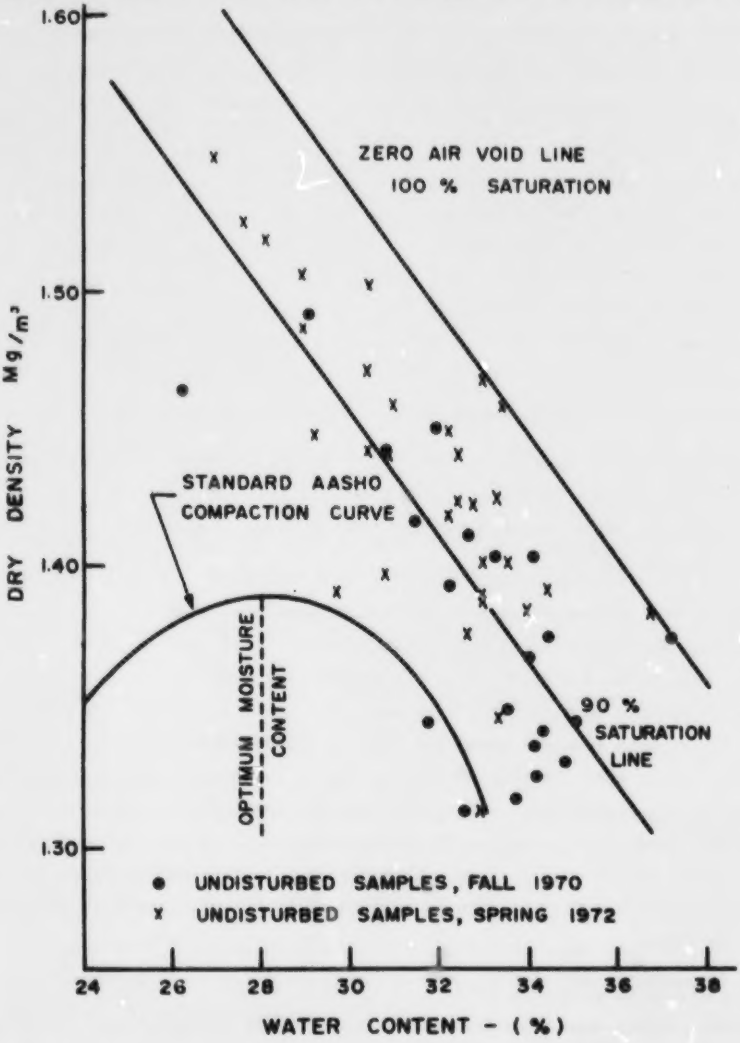

Figure 18/ Water Content vs Dry Density for Regina Clay
(Bergan and Monismith, 1972)

frequently an order of magnitude less (Sauer and Monismith 1968; Bergan 1972).

Field measurement of matric suction throughout the world have been made using a variety of devices. The choice of instruments depends largely on the magnitude of the suction values.

De Bruijn (1965) measured suctions of from 100 to 1,000 kPa under covered areas in South Africa. Richards (1969) reported similar values in a number of areas in Australia. Psychrometers were used in both these areas to monitor suction. The psychrometer has apparently worked reasonably well to measure suctions of this magnitude. Bergan (1972) used psychrometers in an attempt to monitor suctions in a highway subgrade. However, limits of accuracy under these field conditions are in the order of 25% or ± 100 kPa, whichever is greater (Richards 1969).As a result these efforts met with marginal success only. Morrison (1980) suggested an accuracy of ± 40 kPa might be achieved under highly controlled laboratory conditions.

Tensiometers have been used successfully to measure matric suctions of one atmosphere or less (100 kPa). McKim et al (1976) have extended the use of tensiometers below freezing temperatures by using an antifreeze solution that is miscible in water.

Ongoing work at the University of Saskatchewan indicates that a portable tensiometer ("Quick Draw" Soil Moisture Probe)* will provide reasonably good measurements of suction in the field. The probe simply consists of a porous ceramic sensing tip at the base of a small bore capillary tube that is filled with de-aired water and that is connected to a vacuum dial gauge. The probe is approximately 30 cm long and is accompanied by a coring tool that is required to form a hole prior to the insertion of the probe. When the tip comes into contact with unsaturated soil, the soil tries to absorb the water in the porous stone. This places a tension on the water in the capillary tube. This tension is registered on the dial gauge at the top of the tube.

This device is presently being used to monitor matric suction of an unstable railway embankment on the CPR line between Cranbrook and Fort Steele,

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* Soil Moisture Equipment Corp., Santa Barbara, California.

British Columbia. Some of the remedial measures that are contemplated require knowledge of the soil matric suction-depth profile throughout the year (EBA Consultants 1982).

Peck and Rabbidge (1969) recommended an osmotic tensiometer for measuring total suction. The device employs a chamber containing a solution of a high molecular weight compound separated from a saturated porous stone by a semi-permeable membrane. When the probe is inserted into the soil, water tends to flow through the membrane creating a positive pressure in the chamber. The resulting increase in pressure is a measure of the total soil suction. Bocking (1977) performed extensive tests on this device and concluded that it was unsuitable for measuring total suction because of the transient nature of the response of the tensiometer to a change in temperature. In addition long term drift in the calibration indicates that it could not be used as a permanent installation at a field site.

The need for a device still exists that will measure matric suctions within a range of 1 to 3 atmospheres (100 to 300 kPa). Perhaps a fluid may be developed which is miscible in water and that can develop and sustain tensions greater than 1 atmosphere. A compressible porous sensing tip may have some merit by preventing cavitation. The volume change of the tip might be monitored and translated into a matric suction value for the soil.

Continuous and remote sensing of suction is hampered by the need to service tensiometers on an almost daily basis. The fluid in the capillary tube and porous sensing tip frequently cavitate causing the instrument to malfunction.

Continuing Research

Limiting deflection of pavement surfaces remains the most effective method of increasing the expected life of a pavement surface. As the costs of energy increase, the trend is towards heavier axle loads to maintain or limit the increase in transportation costs. As this course of action continues and as the cost of maintaining a highway system escalates, it is extremely important to explore all possible methods of reducing pavement deflections and extending pavement life.

Ongoing research at various government agencies and universities in North

America and elsewhere continue to provide more reliable testing equipment to measure the relevant parameters during dynamic loading, more powerful analytical analyses, and a greater understanding of the freezing phenomenon.

The placement condition of the subgrade and its possible effect on in-service suction level is one important concept in pavement management. The preservation of the soil moisture suction at the time of construction and the concomitant soil deformation properties may be realized by encapsulating the subgrade in a membrane, by using insulation to reduce frost penetration, or even by more attention being paid to drainage and snow accumulation. Some of these techniques along with a better understanding of soil matric suction changes, from not only one season to another, but from one highway section or location to another, are important concepts for controlling subgrade behavior. This is an area of research that has the possibility of significant return in extending pavement life or reduced annual cost of providing an acceptable pavement surface.

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Session 2

Design, Construction and Frost Susceptibility of Lime Stabilized Marine Clay in Highway Subgrade Fill

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ABSTRACT

A large quantity of wet, soft, silty clay was modified with a low (1.5 to 2%) mixture of hydrated high calcium lime in conjunction with freeway cut/fill grading operations in the Ottawa area. The natural clay derives from a marine saline deposit known locally as Leda Clay. In the project area the in situ moisture and sensitivity of the clay were such that it was unsuitable for conventional roadway fill construction.

During construction, the addition of lime dramatically strengthened the soft clay, allowing conventional production rates of fill construction within the problem area. The modified clay was placed up to the subgrade level. Excellent fill stability and compaction conditions were achieved. In order to strengthen the subgrade through the cut areas, a 500 mm depth of the subgrade was modified with a higher (5 to 6%) lime content. This section was insulated with a 25 mm thickness of polystyrene board in consideration of the frost susceptibility of the underlying poorly-drained, soft, silty clay subsoil.

In the first winter after completion of the pavement and opening to traffic, the sections over the modified fill construction developed severe distortions due to differential frost heaving. The magnitude of the distortions was such that the driving lanes of the freeway had to be closed for a considerable period.

This paper outlines the design-site and laboratory-investigation analysis, the construction procedures and test data, the performance of the roadway including related climatic data, the site investigation during and after the frost heaving, remedial measures to restore the roadway and follow-up laboratory testing and evaluation to determine causes of the unexpected frost action. Detailed test procedures and data are provided. As well, the paper discusses and recommends design and construction procedures to be used on projects with similar soft clay problems where lime modifications may be an advantage. Direct frost heave measurement tests on prepared specimens of the modified soil proposed for fill construction are recommended.

DESIGN, CONSTRUCTION AND FROST SUSCEPTIBILITY OF LIME STABILIZED MARINE CLAY IN HIGHWAY SUBGRADE FILL

1/ INTRODUCTION

A very wet, soft marine clay was modified with a small percentage of hydrated high-calcium lime and subsequently placed in freeway subgrade fills in the Ottawa Area. After some trial and adjustment with construction equipment and operations, success was attained by the contractor in achieving high-quality, well-controlled fill construction. The use of lime treatment involved considerable construction savings by reducing the disposal requirements of the soft plastic clay and the quantity of imported fill required to construct the fill sections. In the winter and spring following completion of the pavement and opening to traffic, severe differential frost heaving occurred. Traffic lanes had to be closed, with traffic confined to one lane on the passing side of each of the divided roadways. After the spring thaw, substantial corrective and preventive work was required to restore the roadway.

This paper outlines the laboratory test procedures to determine the stabilizing effects of the lime on the soft clay, construction procedures and conditions involved with the treatment and cut-fill operations, and subsequent site and laboratory investigation to determine frost heaving of soft clay treated with various percentages of high-calcium lime. Remedial measures taken to rehabilitate the roadway are also outlined.

2/ DESIGN

Pre-engineering samples of the in situ proposed cut material indicated a a clay of intermediate to high plasticity. The materials were in a very wet condition with natural water contents at or near the liquid limit. Atterberg Limits and natural water contents are indicated in Table 1.

It is not practical to place these materials in fill as far as normal construction practice is concerned.

Three tests were carried out to determine the effects of lime addition on the soft clay material. These were California Bearing Ratio, Atterberg

Limits and hydrometer analysis to determine the grain-size distribution of the material.

2.1/ California Bearing Ratio

Details of the California Bearing Ratio (CBR) test procedures may be found in ASTM Standard D 1883-73. Samples of the material are well mixed with the required lime percentages and cured in plastic bags for 24 h before compaction in the CBR moulds. This simulates practical construction procedures of mixing the lime into the clay and allowing curing or strengthening of the material before removing and placing in fills. The compacted CBR specimens are soaked for four days before penetration testing is carried out.

Approximate data collected from the CBR Tests are shown in Table 2 and the penetration curves are shown in Figure 1.

2.2/ Atterberg Limits

Details of the test procedure may be found in the Ontario Ministry of Transportation and Communications (MTC) Testing Manual (LS-703 and LS-704).

The addition of lime has a marked effect on the Atterberg Limits. Both the liquid limit and the plastic limit increase with increasing lime content; however, the plastic limit increases more than the liquid limit; hence, the plastic index or range of plasticity is decreased.

The tests were carried out by adding small percentages of lime to specimens of the natural soil in increments of 1 to 2%. The lime-soil mixture was then cured for 24 h in plastic bags before performing the normal test procedures. The plastic limit increases with the addition of lime to a point where further increments show little change. This is known as the "fixation point" [Hilt and Davidson (1960)]. For the material tested, the "fixation point" was found to be approximately 1.8% with high-calcium and dolomitic lime.

2.3/ Grain-Size Distribution

The grain-size analysis tests on the 24 h cured lime/clay mixtures showed a coarsening effect with lime addition increments. This is due to the agglomeration of the small clay-size particles forming silt and sand-size particles, hence, changing the apparent particle-size distribution of the soil mass. Figure 3 illustrates the grain-size distribution of the natural clay and lime/clay mixtures after 24 h curing.

This test is utilized to identify the frost susceptibility of untreated soils for design purposes. MTC criteria to identify frost susceptibility of a soil mass are as follows:

Percent Grain Size Between 5 µm & 75 µm	Frost Susceptibility
0 - 40	Low
40 - 55	Moderate
55 - 100	High

Both the natural and treated soils indicated low frost-susceptibility characteristics by these time-tested criteria.

3/ CONSTRUCTION

The successful contractor's bid, indicating an approximate cost saving of $200 000, was achieved by treating the soft clay with 2% lime and placing it in the fills, rather than wasting it.

Construction was started in the fall of 1976 but was primarily carried out during the summer months of the following year. The contractor initiated hauling the lime from the supply source in large truck-mounted tanks with rear cyclone-spreaders attached. A self-powered pulvi-mixer was next brought to the grade to mix in the material. Both of these units bogged down in the underlying soft clay and progress was impeded to the point that alternative equipment was brought to the job. Lime application was made with large 15 m^3 self-propelled rubber-tired earth scrapers (Figure 4). After a few trial runs, uniform, controlled spreading of the lime was obtained through a narrow discharge opening over the scraped width while the scraper travelled at a uniform speed (approximately 15 km/h). After a short waiting period, the material was then mixed into the clay with a 90 cm diameter tractor-hauled disc. The discs were set at a slight angle during the first pass and the angle was increased as stability improved in order to achieve more plowing and mixing of the lime and clay (Figure 5). This process achieved a uniform mixture and broke down the clods to a clayey, sand-like texture after four to five passes. The treated depth of loose material was approximately 40 cm. After mixing, the material was allowed to cure and strengthen until field observations indicated that it could be excavated and compacted in fills with the conventional equipment. This usually took 6 to 8 h. The treated material was then excavated with scrapers and placed in compacted lifts of 15 to 25 cm. A 30 cm depth of material was removed, leaving 10 cm of treated material to carry the scraper for the subsequent treatment layer. Compaction of the fill lift was carried out with a 25 t self-propelled segmented drum, a "Pack-All" Unit (Figure 6). The lime-soil mixture demonstrated remarkable stability under the heavy compaction unit. The following compaction data were obtained from continuous testing of the fill operation.

Laboratory Target Density (Standard Proctor Test)	
Maximum Wet Density	2.04 t/m^3
Maximum Dry Density	1.69 t/m^3
Optimum Moisture Content	21%

Site Tests	
Field Wet Density	1.80 to 2.01 t/m^3
Field Dry Density	1.53 to 1.69 t/m^3
Field Moisture Content	16 to 23%
Percent compaction	91 to 100%

Most of the density tests ranged close to 95% of the target value. Density tests of the in situ clay indicated 1.64 to 1.87 t/m^3 wet density and 1.29 to 1.45 t/m^3 dry density with 30 to 50% moisture content. This indicates an approximate 15% densification of the solids. Moisture loss during the treatment and grading ranged from 10 to 25%.

This degree of densification was not expected and no densification was estimated during the design. It may be attributed to the moisture loss during treatment and subsequent grading exceeding moisture loss in the laboratory mixing and preparation of specimens. Sunny, warm and dry weather prevailed during most of the operation. The rate of stabilization was impeded by cool, wet weather. When the temperature was below 5°C, progress was slowed to the point that the contractor suspended operations.

The grading operation produced stable, uniform, homogeneous subgrade conditions. The subgrade treated with 5% lime in the cut (45 cm thickness) was quite stable. This section was insulated prior to application of the granular base in consideration of the potential differential frost action within the soft clay under the treated material.

Granular base and paving was carried out under a follow-up project during 1978. The granular base was 15 cm in thickness and consisted of well-graded < 22.4 mm nominal-size material. Bituminous pavement was 28 cm in depth and consisted of dense, graded hot-mix meeting standard Marshall Mix requirements as per MTC specifications. Paving was completed in early fall and the roadway was open to traffic. The paving contract was of a high quality, providing an excellent ride when the roadway was opened.

4/ PERFORMANCE

Excellent pavement conditions were in effect until substantial below-freezing weather occurred. Towards late December, 1978, some differential frost heaving was noted on the lime-treated fill sections. These conditions seemed to stabilize after December and the slightly uneven conditions prevailed through January and February, 1979. Continuous cold weather prevailed through this period. In the first week of March, during a sudden warm period accompanied with rainfall, substantial differentail frost heave distortion occurred over a two-day period. Heave differentials as high as 20 cm developed in the driving lane over lengths of 6 to 7 m (Figure 7). The distortions were mainly located along the shoulder and outer pavement edge, extending into the midlane approximately 1.5 m from the edge of the pavement (Figure 8). Large cracks developed in the gravel surface of the shoulders.

Because the passing lane underwent considerably less distortion, it was possible to divert traffic there while the driving lanes were barricaded off. This condition lasted for two months while major rehabilitation work was done before normal traffic operations were restored.

The frost heave distortions did not occur through the cut sections with insulated subgrade.

4.1/ Weather

Near-continuous freezing weather commenced around November 18, 1978. During December, 1978, and January-February, 1979, persistent cold weather conditions prevailed. From November 18, 1978, to February 28, 1979, the freezing index totalled 1042.3 Celsius-degree days. A relatively small amount of freezing occurred during March.

Table 3 outlines the freezing index and precipitation through the period of frost heave occurrence. Figure 9 illustrates the daily mean temperatures during November, 1978, to March, 1979.

5/ FROST HEAVE, SITE INVESTIGATION

Test pits placed at the pavement edge and within the shoulder area indicated substantial ice lenses in the lime-treated subsoil. The lenses varied in thickness from 0.5 cm to 4 cm and were located at various 2 cm to 15 cm intervals to depths of 1.5 to 1.8 m below the roadway surface.

Samples of the treated subgrade material were obtained. In addition, samples of the soft, untreated clay were taken for follow-up laboratory test and evaluation work.

6/ REMEDIAL MEASURES

Remedial measures to restore the roadway after the thaw period involved installation of subgrade drainage and sealing the shoulder surfaces by asphalt paving.

Subdrains were installed along the edges of pavement 1.2 to 1.5 m below the surface. The drains were 100 mm diameter slotted plastic pipe wrapped with a geotextile filter fabric. The drains were placed in 300 mm wide trenches bedded and backfilled with a clean, fine sand. The sand backfill contained 2 to 4% pass the 75 µm sieve. Outlets to the roadside ditches were spaced at approximately 100 m intervals.

During warm weather in late May, the distorted asphalt-pavement surface was rolled with a heavy, 25 t self-propelled steel roller. After two to three passes, much of the heave distortion was reduced; however, the surface was still distorted from its original condition and the ride over the distorted areas was quite uneven and rough. The rolling treatment was followed up with milling an average 40 mm depth from the distorted sections. The areas were then resurfaced flush with the adjacent undistorted pavement. The milling and repaving varied in width from 1 to 2.5 m from the pavement edge. In conjunction with the repaving, the full shoulder widths were repaved over the entire length of the areas with the treated subgrade material.

These procedures pretty well restored the pavement ride and reduced the magnitude of the frost heaving in the winters subsequent to the restoration. However, frost heaving continues to be a problem over the areas, and additional preventive treatment will be required in conjunction with future overall rehabilitation projects. This will likely involve expensive insulation installations.

7/ LABORATORY INVESTIGATION

After review of the original test data, and consultations with several engineers having considerable knowledge and background in frost action within soils, it was decided to carry out laboratory investigation with various contents of lime mixed with samples of the soft, silty clay from the project site. Samples of the treated fill from the project were included.

7.1/ Equipment

The frost heave apparatus was constructed at the Ontario Hydro Research Division laboratories. It consists of a box fabricated from 13 mm thick plywood and lined with 25 mm thick polyurethane foam sheet on the bottom and to a height of 190 mm on the sides. Figures 13 and 14 give details of the construction.

The metal tray fits inside the foam lining and has square acrylic blocks supporting a wire-mesh plate. A number of Pyro-Tenex heating cables are strung the length of the tray between the acrylic blocks.

Three additional layers of polyurethane foam are fitted on top of the tray, and 11 holes are made in a symmetrical pattern to accept the specimen containers.

The specimen containers are fabricated from acrylic tubing with 6 mm thick walls, and are 200 mm long. A perforated acrylic baseplate fits snugly into the bottom of each specimen container. A disc of geotextile material is used to separate each specimen from the baseplate.

Changes in height of the specimens are measured by means of a dial gauge mounted on a bar which rests on top of the acrylic cylinder.

The whole frost box assembly containing samples is placed in a suitable freezer. Sufficient water is poured into the tray to keep the level approximately 25 mm above the bottom of the test specimens. The water is kept at approximately +4°C and the freezing temperature at approximately -17°C to maintain a one-directional temperature gradient across the specimens. Figure 15 shows the frost box containing specimens in the freezer chest.

The test specimens are compacted in a regular 0.944 dm^3 Proctor mould and extruded. Each specimen is wrapped in a layer of petroleum-jelly-coated plastic wrap. The outside of the wrapped specimen is then coated with petroleum jelly and it is placed on the geotextile disc resting on the perforated acrylic baseplate. The acrylic cylinder is lowered over the specimen and the annular space between the specimen and cylinder is sealed with a large bead of petroleum jelly, the placement of which is facilitated by a large plastic syringe. This is to prevent the specimen from freezing to the cylinder and causing inaccurate height-change measurements.

During the testing, dial gauge readings are taken of height changes of the specimens at convenient time intervals. These are plotted as heave in millimetres vs. time in hours; a typical set of results is shown in Figure 10. The hatched area between 12.7 and 17.8 mm represents the marginal frost-susceptibility classification according to the Transport and Road Research Laboratory (TRRL) in England (Jacobs, 1965). Readings below 12.7 mm are considered satisfactory and readings above 17.8 mm are considered unsatisfactory. All these ratings are taken at a 250 h time-point for uniformity. Figure 11 shows the comparison in heaving magnitude for specimens with lime contents ranging from 1 to 6% and Figure 12 illustrates the amount of heave in the natural soil.

After a completed test, all specimens are photographed with a standard-size (110 mm high) compacted specimen which provides a visual observatio of the magnitude of heaving.

7.2/ Test Programme

Three bulk samples of material were submitted from the site for testing: two in the natural state and one lime-treated material from the fill area. The samples were shipped in plastic bags contained inside normal sample bags in order to retain the natural water content.

On each natural material, duplicate specimens were compacted with high-calcium lime contents of 1, 2, 4 and 6%, based on dry mass. The procedure briefly outlined in Section 2 on design parameters was followed, with the exception that a Proctor mould was used. After compaction, the specimens were stored in plastic bags in a constant humidity environment for about six weeks to simulate field aging.

Series 1

The first natural material had a water content of 31.2%, and the moulding data are given in Table 4.

Series 2

A set of three samples was prepared from the treated material, and the moulding data are shown in Table 5. The compaction and curing was the same for these specimens as for the first series.

Series 3

A third set of samples was prepared from the second natural material which had a water content of 68.9%, considerably higher than the first material. The moulding data are shown in Table 6. Figure 12 shows the pair of frozen samples at 1% lime against a control specimen and also a close-up showing the soil as plate-like flakes in a matrix of ice.

Series 4

A fourth set of samples was prepared from the first natural material ($w = 31.2\%$) and prior to the frost heave tests, and was allowed to take up water for a period of 24 h. The specimens were allowed to stand in the apparatus before being placed in the freezer chest, and the ends of the specimens were immersed about 25 mm below the water level in the tray. In addition, two specimens were prepared from the second natural material (w = 68.9%). These could not be compacted, as the material was too soft but were kneaded into the Proctor moulds before being taken out.

Table 7 gives the relative heaving magnitudes for the three materials in numerical form (mm), with some approximations having been made for specimens where the amount of heave was great. All the submitted materials were in the CI-CH range, the difference between the two natural materials being the water content. The material with the lower water content (31.2%) showed heaving magnitudes approximately half of those for the material with the higher water content (68.9%), except at 6% lime. This is confirmed to some extent by comparing the heaving for the material at a lower water content unsoaked and soaked prior to freezing. The soaked samples showed a heaving magnitude approximately twice that of the unsoaked samples, except at 6% lime.

The natural material at a higher water content heaved less than when 1 and 2% lime were added, and this is consistent with the change in grain-size distribution shown in Figure 3, which indicates that these materials become coarser with the addition of lime. This leads to an increase in permeability, and probably a faster capillary rise in the specimens with 1 and 2% lime before freezing occurs. At higher lime contents, the increased bonding or cementing action of the lime with clay particles probably offsets the hydraulic pressures generated by freezing. In addition, the more open pore structure may assist in the dissipation of hydraulic pressures.

The field-treated specimens showed a magnitude of heave approximately the same as that for the natural material at the lower water content and 1% lime, which would indicate that the field lime content is of the same order.

It would appear from the limited testing carried out that the water content of the material before modification or stabilization is of great significance. Also, the addition of 1 or 2% lime, sufficient to increase the CBR of the material to a marked degree, is of little benefit if the material will be subjected to freezing action. It is of some significance that all the specimens treated with 6% lime gave less than the 12.7 mm minimum readings recommended by the TRRL as being in the satisfactory range.

8/ DISCUSSION

The laboratory frost heave testing and the performance of the project indicate that laboratory frost-susceptibility criteria for natural soils are not applicable to chemically-treated materials. The 1 and 2% lime and clay mixtures appear to have drainage properties and capillarity comparable with natural silt (5 to 75 µm grain-size soil). This phenomenon is probably due to cementation of the clay-size particles forming such pores with the agglomerated soil mass. The continued addition of lime appears to have increased cementation and sealed porosity. The reduced internal drainage reduces water access to the soil mass, thereby reducing the main ingredient for ice lense formation. The increased strength from the added cementation further increases soil deformation resistances and further reduces the frost susceptibility of the soil mass.

The laboratory test procedures utilized in this project to evaluate the prepared lime/clay specimens and the sample of treated material from the project appear effective in simulating the environmental effects of moisture and frost on the chemically-treated materials.

In that lime content is directly related to the frost susceptibility of lime/clay mixtures, close control of the lime content is of prime importance during construction. This should be done by constant checks of the lime distribution and by site titration tests as the work progresses. The titration tests should be carried out in accordance with ASTM standard D3155-73. This test involves establishment of calibration curves in a properly-controlled soils laboratory environment. The site test values are applied to the calibration curves to determine the lime content of the sample.

9/ CONCLUSION AND RECOMMENDATIONS

- The addition of small quantities of hydrated lime was most effective in stabilizing the soft clay material. The CBR and Atterberg Limits tests provided practical and accurate evaluation of the stabilization that occurred during construction. These tests should be continued for design evaluation in circumstances where stabilization of soft clay may be advantageous.

- The grain-size distribution test of chemically-treated specimens may provide an indication of stabilization; however, it should not be utilized by itself to assess frost susceptibility.

- Frost heave testing should be carried out to determine frost susceptibility of chemically-treated soils. The testing should involve exposure in a freezing chamber with a continuous water supply. This test does not cost much and can be carried out in a practical time frame.

- Treatment of soft problem clays with various small proportions of lime should continue to be considered in areas affected by frost action. However, the aforementioned frost heave test should always be carried out as a part of the evaluation. Tight construction control of the lime content must be adopted in any lime-treatment project.

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Table 1/ Engineering Properties of the Materials

Natural Water Content	(w)%	45-52
Liquid Limit	(W_L)%	46-57
Plastic Limit	(W_P)%	17-25
Plastic Index	(I_P)%	29-32
Lime Fixation Point %		1.5-2

Table 2/ CBR Design Data

	Water Content %	Plastic Limit %	Dry Density tonne/m^3	CBR
Natural Material	49	21	1.14	0.5
1% lime	43	37	1.20	4.6
2% lime	45	40	1.18	7.5
4% lime	38	40.5	1.31	9.0

Table 3/ Freezing Index and Precipitation

Date	Freezing Index (Celsius-degree days)	Precipitation	
		Rain (mm)	Snow (cm)
Nov. 78	72.4	47.1	24.7
Dec. 78	251.7	26.2	67.1
Jan. 79	316.4	19.9	86.2
Feb. 79	401.8	5.6	25.5
Mar. 79	3.2	39.6	10.4

Table 4/ Moulding Data (Series 1)

	Specimen No.			
	1 & 2	3 & 4	5 & 6	7 & 8
Lime Content (%)	1	2	3	4
Wet Density (t/m^3) Av.	1.908	1.832	1.817	1.766
Water Content (%) Av.	30.5	32.2	31.5	26.9
Dry Density (t/m^3) Av.	1.463	1.386	1.381	1.391

Table 5/ Moulding Data (Series 2)

	Specimen No.		
	1	2	3
Lime Content		Field	
Wet Density (t/m^3)	2.019	2.019	2.019
Water Content (%)		22.2	
Dry Density (t/m^3)	1.652	1.652	1.652

Table 6/ Moulding Data (Series 3)

	Specimen No.			
	1 & 2	3 & 4	5 & 6	7 & 8
Lime Content (%)	1	2	3	4
Wet Density (t/m^3) Av.	1.654	1.654	1.673	1.692
Water Content (%) Av.	54.8	55.6	51.5	49.7
Dry Density (t/m^3) Av.	1.069	1.061	1.104	1.130

Table 7/ Heaving Magnitudes in Millimetres

Lime Content (%)	0	1	2	4	6
			Heave in mm		
First Material (low w)	—	50	25	5	4
Field-Treated Material	—	58*	--	-	-
First Material (soaked)	—	135	45	10	5
Second Material (high w)	122	127	127	38	5

*Lime content assumed

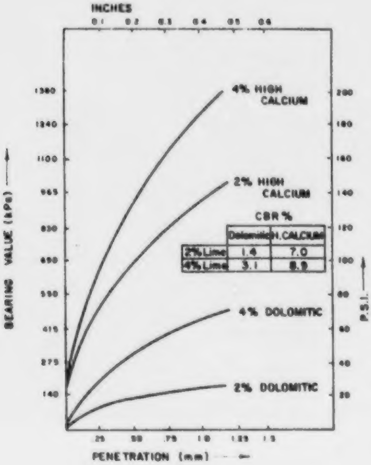


Figure 1/ CBR Test Values of Lime/Clay
Mixtures after 24 h Curing

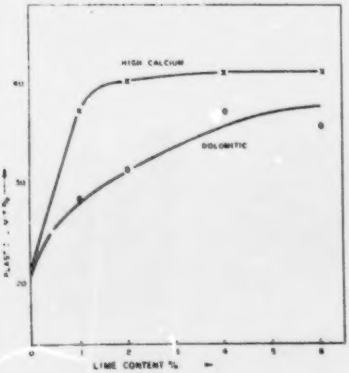


Figure 2/ Plastic-Limit Test Values with
High-Calcium and Dolomitic
Lime Mixtures after 24 h Curing

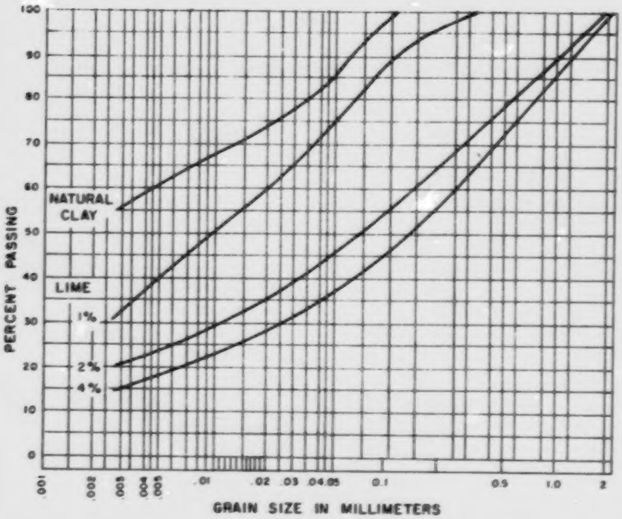


Figure 3/ Grain-Size Distribution of High-Calcium/Clay Mixtures after 24 h Curing

Figure 4/ Excavation of Lime-Treated Clay

Figure 5/ Disc Mixing Lime into Clay

Figure 6/ Compaction on the Lime/Clay Fill

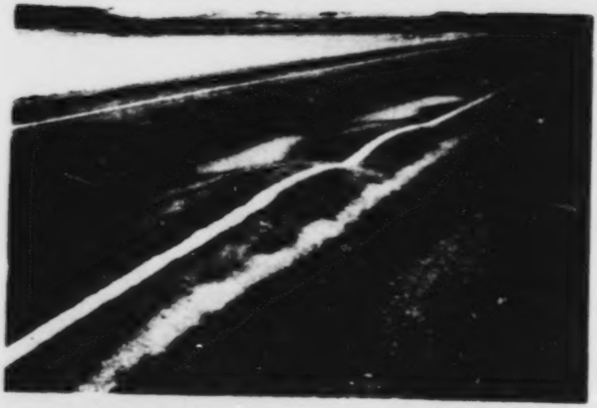


Figure 7/ Frost-Heaved, Granular Shoulder and Asphalt Driving Lane

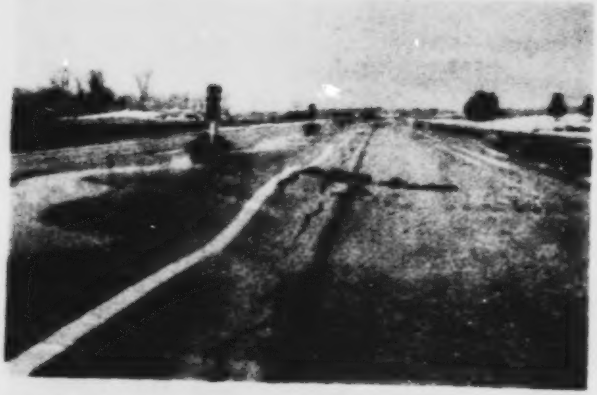


Figure 8/ Traffic-Control Barricades along Heaved Area

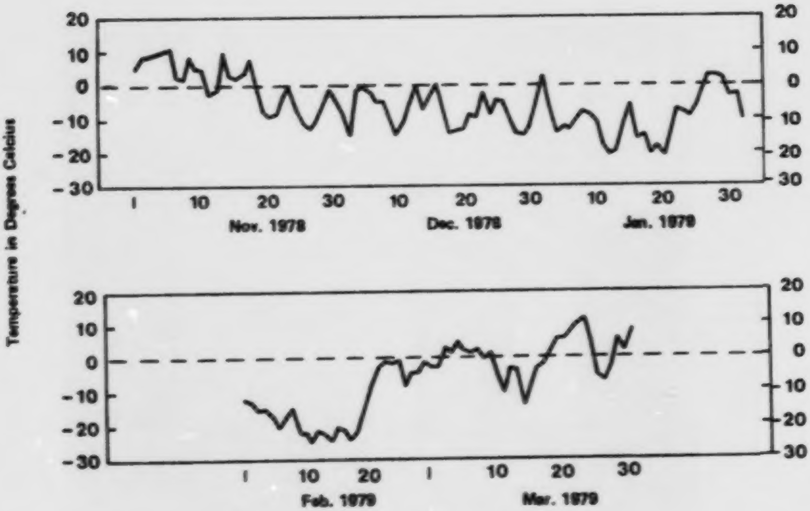


Figure 9/ Daily Mean Temperatures - Nov. 1/78 to Mar. 31/79

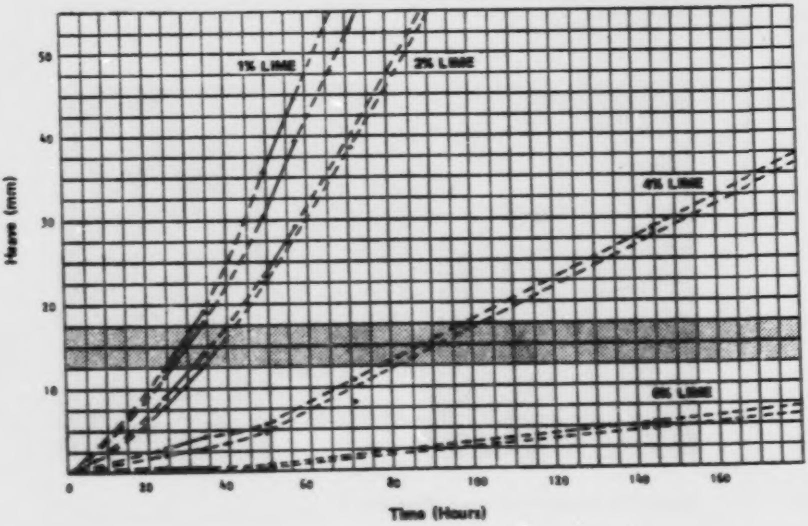


Figure 10/ Frost Heave (Series 3).

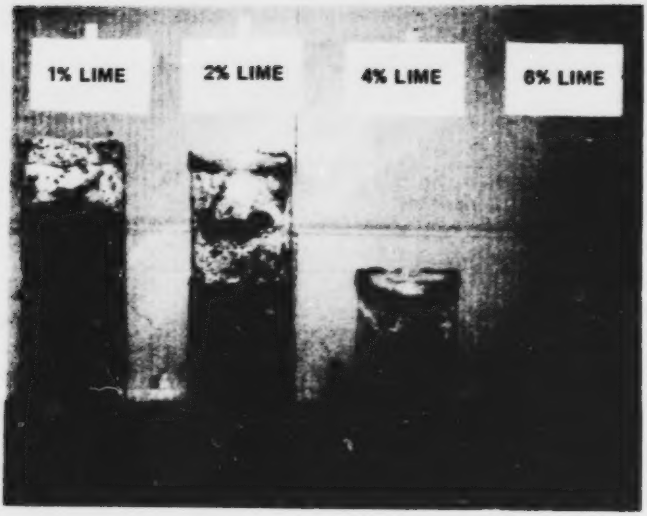
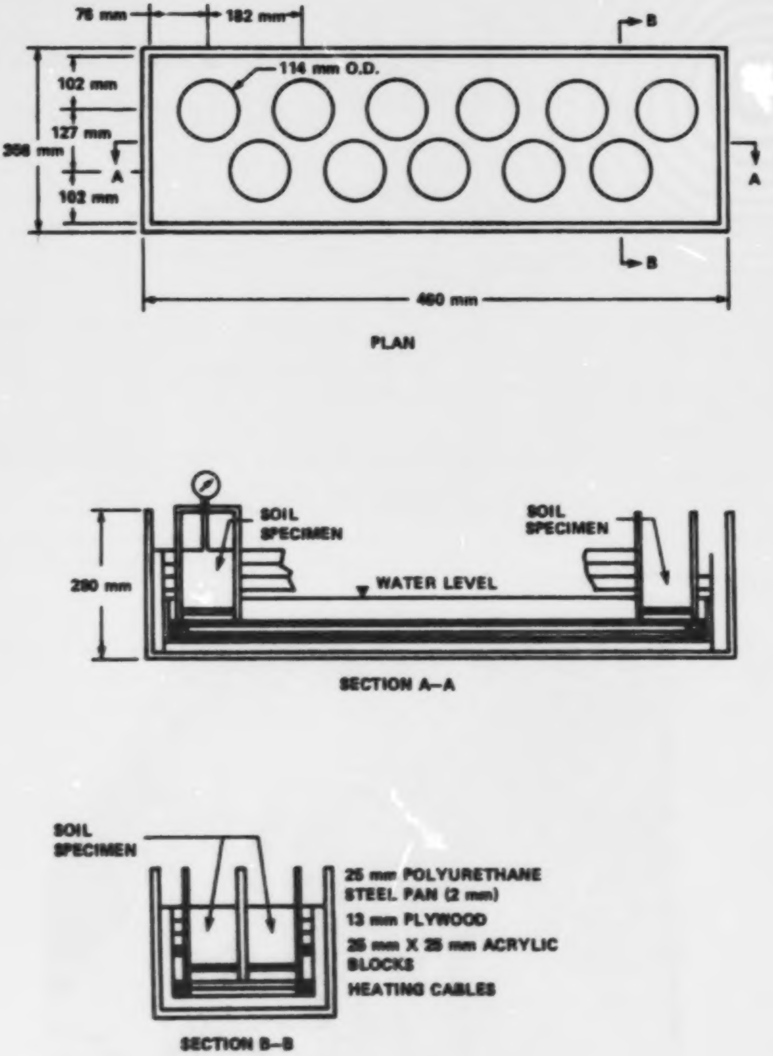

1% LIME
2% LIME
4% LIME
6% LIME

Figure 11/ Comparison of Series 3 Specimens

CONTROL
CLAY ONLY

Figure 12/ Frozen Specimens (Clay Only)


76 mm
182 mm
114 mm O.D.
102 mm
127 mm
356 mm
102 mm
A
A
B
B
460 mm
SOIL SPECIMEN
SOIL SPECIMEN
WATER LEVEL
290 mm
SOIL SPECIMEN
25 mm POLYURETHANE
STEEL PAN (2 mm)
13 mm PLYWOOD
25 mm X 25 mm ACRYLIC BLOCKS
HEATING CABLES

PLAN

SECTION A–A

SECTION B–B

Figure 13/ Frost Heave Box

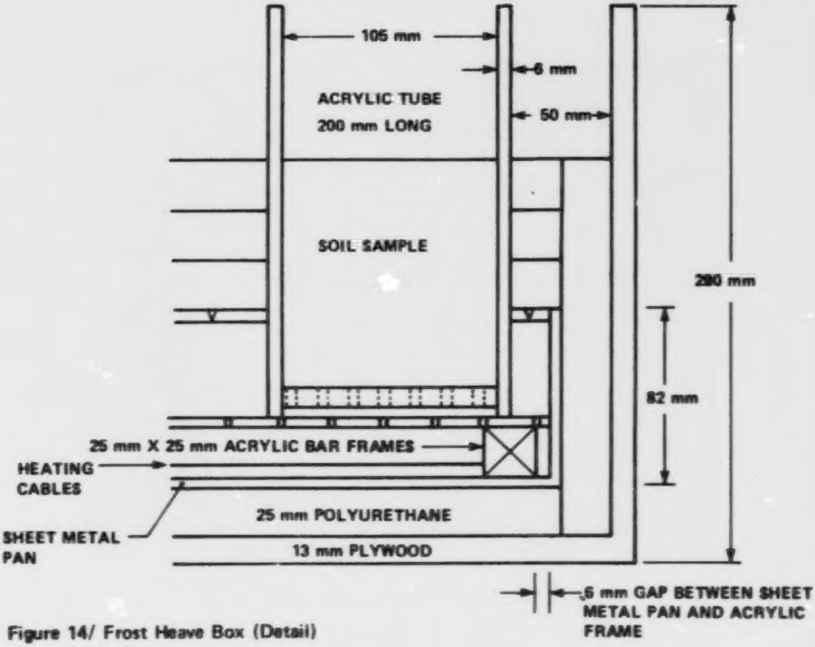

105 mm
6 mm
ACRYLIC TUBE
200 mm LONG
50 mm
SOIL SAMPLE
280 mm
82 mm
25 mm X 25 mm ACRYLIC BAR FRAMES
HEATING CABLES
SHEET METAL PAN
25 mm POLYURETHANE
13 mm PLYWOOD
6 mm GAP BETWEEN SHEET METAL PAN AND ACRYLIC FRAME

Figure 14/ Frost Heave Box (Detail)

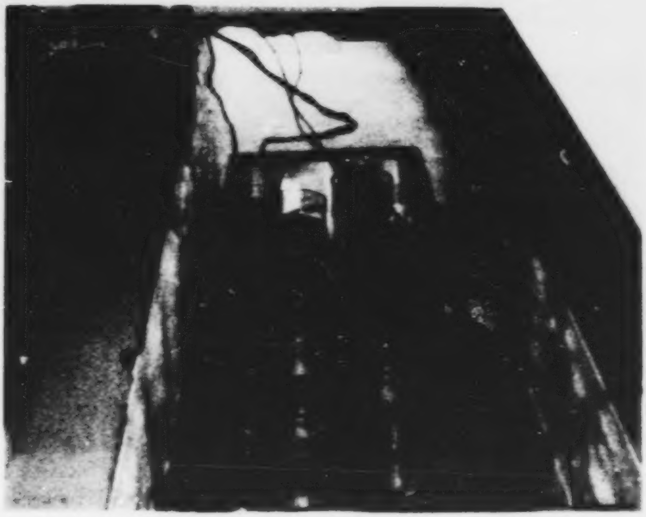


Figure 15/ Frost Heave Box in Freezer Chest

Session 3

Measurement and Prediction of Frost Penetration in Highways

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ABSTRACT

Knowledge of the depths of frost penetration in highway roadbeds is essential to determining the safe location of structural foundations, sewers, pipelines, and watermains. To a great extent, current knowledge of frost penetration is the result of sparsely-documented experience gained from studies on boreholes and excavations, although a method of prediction exists which relates frost penetration to air freezing index.

Through a five-year program of measuring frost depth by use of frost depth indicators at 62 locations in Ontario, a body of data has been acquired and used to make the frost penetration versus air freezing index relationship more appropriate to Ontario. To further assist the user of this relationship, a freezing index map of the province has been created from published temperature data, observed at 224 locations during the period from 1941 to 1970. The map was prepared using a method recently developed by the National Research Council of Canada which calculates the air freezing index from mean monthly temperatures.

While the method is simple to use, certain local factors such as soil type and moisture content influence the actual depth of frost penetration. To enable such effects to be evaluated, and to provide the capability of examining the effects of insulation and frost retaining layers, a computerized solution of the heat flow relationships was developed for the Ministry by Carleton University. The program is able to predict the maximum frost penetration and ground temperatures from input information which is, in most cases, readily available. Computer predictions at several sites have been favourably compared with measurements of actual frost penetration/time profiles.

MEASUREMENT AND PREDICTION OF FROST PENETRATION IN HIGHWAYS

1/ INTRODUCTION

Winter temperatures in Ontario often reach -50°C and remain for long periods at temperatures of less than -30°C in many parts of the province. In addition, many areas of Ontario experience more than 150 days of below freezing weather each year. Under these conditions, freezing temperatures penetrate from 1.5 m to 3 m below highways and consequent volume changes due to frost action result in pavement surfaces heaving upwards in spots as much as 0.3 m. Frost heave threatens the integrity of roadway structures, sewers, pipelines, and watermains. To prevent disruption of the services provided by these facilities and to avoid structural foundation problems due to frost action, these structures and foundations must be located below the frost line. Since cost and difficulty increase rapidly with the depth of excavation needed to locate the structure below the frost line, a knowledge of the greatest depth of frost penetration is essential.

Until fairly recently, estimates of frost depth penetration were based on undefined past records of boreholes and excavations carried out by various works departments, contractors and other companies. Theoretical calculations based on heat transfer equations had been used by Sanger in 1963 [1] to produce frost depth penetration curves for different soil types, as illustrated in Figure 1. This figure shows a wide-spread between different soils. For example, at 3000 degree-days (F) there is a spread from 1.4 m for clay to 3.7 m for sandy gravel. In addition to these records, other methods of estimating frost depth penetration under highways are now available.

One such method, established by the U.S. Corps of Engineers in 1947 [2] utilizes a relationship between frost penetration and air freezing index. The method was made applicable to Canada by the publication of a freezing index map of Canada by Wilkins and Dujuy [3] in 1954. Further confirmation of the frost penetration/air freezing index relationship was obtained by a Ministry of Transport (MOT) study at airports throughout Canada [4].

This relationship takes the form "P (metres of frost penetration) equals a constant times the square root of F (the air freezing index in degree-days)." The method of determining F is described in Section 7 and is identical to the definition given by the U.S. Corps of Engineers.

A more specific relationship for frost penetration/air freezing index was developed for Ontario highways. A revised freezing index map of the province was created from 30 years of air temperature records taken at 224 locations.

Another method of determining frost penetration, which is becoming more readily applicable because of advances in computers, is through calculation of heat flows given the soil parameters and air temperature regime.

Both methods are examined in this paper and compared with actual frost penetration measurements. The advantages and disadvantages of both methods are discussed.

2/ FROST INDICATORS

The frost indicators used to measure frost depth (Figures 2 and 3) are adapted from the Swedish (Gandahl) design and the U.S. CRREL design. The indicators were made and installed by the staff of the Regional Geotechnical Section, MTC, under the direction of the Engineering Research and Development Branch.

The Gandahl type indicator consists of a rigid transparent plastic tube with blue indicator solution contained in an annular space created by a soft rubber insert in the tube. The dark blue indicator fluid (0.05% methylene solution) turns white when frozen. The indicator tube is buried in the pavement surrounded by a close-fitting, telescoping casing. The length of tube required is estimated from the freezing index at the location of the installation.

To read the depth of frost penetration, the covering cap is unscrewed, the indicator tube withdrawn, and the depth of the frozen white solution measured. The depth of thaw which occurs is also evident in the spring, when the melted indicator solution at the top regains its blue colour.

In the two northern regions of Ontario, a modified design (Figure 3) is used because the lengths of acrylic tubing needed for the indicator tube are usually too short. The rigid acrylic plastic tube with a soft rubber insert was replaced by a soft transparent vinyl tube with a solid nylon insert. The same methylene blue indicator solution, however, is used.

In 1975, an indicator designed and developed by the U.S. Army Cold Regions Research and Engineering Lab was used in three Ontario regions. This indicator consisted of a rigid acrylic plastic tube filled with a saturated mixture of Ottawa sand and a fluorescein dye solution. The dye, a natural lime-green colour, turns brown upon freezing. This type of indicator more closely parallels natural soil conditions and is now used in the southern regions of the province.

After four years of use, several frost depth indicators of the Gandahl type were replaced due to damage, or were made longer so that deeper frost penetrations could be measured.

3/ FROST DEPTH MEASUREMENT PROGRAM

The locations of the frost depth indicators were selected by members of the Regional Geotechnical Section according to subgrade soil types and geographical distribution. During the fall of 1970, 33 frost depth indicators were installed in 15 districts in the five regions of the province. During the next two years, an additional 29 frost depth indicators were installed, increasing the distribution to 17 districts in the five regions. Indicators were installed near district patrol yards for convenience.

Each indicator is identified by a two-number set: the District number separated by a dash from the number of the indicator (expressed in ascending numerical order beginning at 1) in its particular district.

The approximate locations of the indicators are shown in Figures 4 and 5, the freezing index maps of northern and southern Ontario, respectively.

The indicator tubes are maintained by Regional Geotechnical and Section staff, but measurements are taken periodically and recorded by members of District Patrol Yards. Examples of the recording forms which show frost penetration depths measured at different dates are given in Figures 6 and 7.

4/ AIR FREEZING INDEX

Site details for the installations are given in Table 1. The results of the frost penetration measurements taken over five seasons from 1970 to 1975 are summarized in Table 2.

These frost measurements were correlated with the air freezing index calculated from air temperature records at the nearest weather stations.

Sites not situated close to a weather station were not used in the correlation. The freezing index is defined as the number of degree-days (above and below 0°C) between the highest point in the autumn and the lowest point in the spring on the cumulative degree-day time curve for one winter season (Figure 8).

5/ FROST PENETRATION/AIR FREEZING INDEX

During the five years when measurements were taken, Ontario experienced the mildest winters in over 100 years. Weather conditions during the remaining years in which measurements were taken ranged from below- to near-normal temperatures.

Frost penetration depends on climatic factors and soil conditions. However, this study was primarily concerned with the major variable of air freezing index as a basis for estimating maximum frost penetration. Using the data collected from various locations throughout Ontario, the frost penetration "P" in metres for cleared asphalt pavement was plotted against the corresponding air freezing index degree-days below 0°C. The complete set of data gathered during the five-year study was analyzed by a regression analysis program which showed a correlation equation of:

$$P = -0.328 + 0.0578 \quad F$$

where:

P = frost penetration in metres

F = freezing index in degree-days Celsius

This regression equation line is plotted in Figure 9. The negative values at the lower freezing indices indicate that the freezing index must reach a minimum value before freezing in the road structure will occur.

A recently-updated report by the Ministry of Transport [4], covering the period 1964 through 1971, gives a relationship for uncovered asphalt airport pavements as:

$$P = -0.61 + 0.068 \quad F$$

The MTC curve for 1970 to 1975 and the MOT curve for 1964 to 1971 do not differ greatly. Between 300 and 2500 degree-days Celsius, the average difference between the two curves is approximately 0.11 m. This difference would not be significant in pavement design. The MOT curve frost measurements (1973) indicate deeper frost penetrations above 1000 degree-days than the MTC curve. This is most likely due to deeper granular structures used in airports than those normally encountered in highways.

6/ EFFECTS OF SOIL CONDITION AND DRAINAGE

Frost penetration is a complex heat flow problem with a great number of other independent variables. Since the frost indicator study used only air temperature as the main variable, some of the results show inconsistencies which may be partly explained by differences in soil conditions.

Soil type and water content can be said to have a combined effect. Gravel and shot rock subgrades with relatively low water contents permit greater frost penetration than clays, which usually have a much greater moisture content. Sand subgrades tend to fall somewhere in between. Dry soils freeze quickly and deeply, and thaw quickly. A wet, heavy soil freezes slowly and has less frost penetration.

The effects of soil type and water content can clearly be seen from the results taken at Sioux Lookout. Site 20-5, located in silty clay soil with a high water content, is compared to nearby site 20-6, situated in well-drained sandy soil. The results, which show a difference of over 0.6 m in frost penetration, indicate the significant effect of water on the thermal properties of the soil. The effect of water can be partially explained by the fact that greater amounts of water present increase the volumetric heat capacity to such an extent that a large heat extraction is necessary to lower the temperature.

This same effect is evident at the two Bancroft sites (10-1 and 10-3, Figure 4) and at the Thunder Bay locations (19-1 and 19-5, Figure 5). The wet, heavy soils have less frost penetration than nearby sites located in well-drained, lighter soils.

The graph shown in Figure 9, together with the new freezing index map of Ontario (Figures 10 and 11), can be used to give an approximation of frost penetration. The standard error of estimate was found to be 0.23 m.

The variation in frost penetration caused by different soils and moisture conditions as measured in this program is not as great as might be expected from theoretical considerations, as is shown by Sanger in Figure 1. A plot of frost penetration predicted by use of the correlation equation (Equation 1) against measured frost penetration shows 80% of the measured points fall within 0.3 m of the prediction (Figure 12).

As the 62 locations cover a wide range of subgrade types, it appears that for design purposes one can accept a depth calculated by adding 0.3 m to the frost depth penetration predicted from Equation 1 (Figure 13). Perhaps a somewhat greater allowance needs to be made for very thick granular structures.

7/ FREEZING INDEX MAP OF ONTARIO

Ontario covers a large, diverse territory, ranging in the south from the same latitudes as Northern California, and northerly to the subarctic shores of Hudson Bay. The associated temperature regimes and their related depths of frost penetration also vary considerably. Freezing index maps previously used were published by the National Research Council of Canada; however, maps of smaller scale cannot show local variations of freezing index, which can often be significant. Also, since 1975, all temperature data is in degrees Celsius and it was felt that an up-to-date freezing index map using degree-day Celsius was needed.

To construct this map, climatological data for over 200 stations in Ontario available from the Atmospheric Environment Services of Environment Canada [5] was used. The data was used in conjunction with the method described in a recent report by the National Research Council of Canada [6] by which reasonably-accurate calculations of air freezing index can be made using monthly mean temperatures. This method, briefly described as follows, was used to calculate the normal freezing index of stations in Ontario which were not included in the report published in 1973 [7].

7.1/ Method of Calculation

Since soil freezing depends largely on the extent and duration of depressed air temperatures, it is relatively easy to measure time and temperature by degree-days. One degree-day is defined as the difference of 1°C of the mean daily temperature from the freezing point of water (0°C or 32°F).

These negative differences each day are accumulated over the winter months. Any day that has a mean temperature above freezing (a positive difference) will reduce the accumulated freezing degree-day total. A cumulative plot of degree-days versus time results in a curve similar to Figure 8.

Since the data is accumulated, it is not necessary to begin the plot on any particular date. The freezing index is the number of degree-days between the maximum point in the autumn and the minimum point in the spring.

Previous to 1975, all temperature data was recorded in degrees Fahrenheit and the method for estimating freezing index was based on this data. The following calculation, therefore, uses degrees Fahrenheit.

Any spring or fall month in which the seasonal cumulative degree-days reach a maximum or minimum, as shown in Figure 8, is called a changeover month. The freezing index for all other months is calculated simply by:

$$N\ (32°F - T)$$

where:

N = the number of days in the month

T = the mean daily temperature

For changeover months, Boyd [6] proposed an equation of a hyperbola of the form:

$$y^2 - y\ (t - 32) = k^2$$

If N is the number of days in the month, then the thawing degree-days are given the positive value of y times N and the freezing degree-days by the negative value of y times -N. The value of k is recommended to be 2.5.

The total seasonal freezing index was found by adding together each monthly total, including the changeover months. The error of estimate for normal degree-days based on a 30-year monthly mean temperature would be about nine degree-days Fahrenheit in the annual totals. However, it should be noted that, because of differences in elevation and topography, nearness to large population centres, proximity to large bodies of water or other sources of heat, variations may be found in annual air temperature of as much as 1°F or 2°F at stations only a few kilometres apart. This could lead to a difference of as much as 100 to 200 degree-days Fahrenheit in the air freezing index over a relatively small geographical area. Thus, no matter how accurate readings are from any one station, a probable error of up to 100 degree-days Fahrenheit is likely for sites within only a few kilometres. Such variations may be significant enough to affect a pavement design, especially in areas with a low freezing index.

7.2/ Constructing the Map

The normal freezing index for the climatological stations in Ontario not published was calculated in the above manner from monthly data supplied by the Atmospheric Environment Service of Canada.

The calculated normal freezing index for all 224 stations in Ontario was plotted on two large-scale maps of northern and southern Ontario. The location and freezing index for each station was converted to digital form and the data changed to degree-days Celsius by applying a factor of 0.5555. Several key points on the map were digitized to enable subsequent plots (overlays) to be accurately transferred back to the original maps.

An IBM 375 computer connected to a plotter was used to interpolate the points and draw isopleths at intervals of 100 degree-days Celsius for southern Ontario and at 200 degree-days Celsius for the northern Ontario map. The maps of Ontario and the isopleth plots were then matched and redrawn as shown in Figures 10 and 11. These maps are sufficiently detailed to enable the designer to establish his site location, and to determine an appropriate freezing index.

A test of accuracy of the map was carried out by reading off the freezing index at each frost depth indicator location, and comparing it with the observed freezing indices during 1970 to 1975.

The comparison, plotted in Figure 14, indicates that the majority of observations fall within 300 degree-days of the map values. The winter of 1973/74 was the coldest in the last 10 years, and 1974/75 was a very mild winter. The map, therefore, reproduces mean conditions very well, and can be used with a fair degree of confidence.

8/ FROST PENETRATION PREDICTION BY COMPUTER SIMULATION

The traditional, analytical estimation of frost penetration is frequently based on a number of empirical relationships and formulae developed over the years. Such methods utilize either air or pavement freezing index but take no account of the other key variables, such as soil type and moisture content.

The most common method used is the modified Berggren [9] equation, which has been adapted to layered systems such as those found in pavement structures. However, these analytical solutions have weaknesses and restrictions, such as the simplifying assumption that all materials have homogeneous soil properties, or that all soil moisture freezes at 0°C. An alternative approach for predicting frost penetration is by computer simulation.

A project recently carried out for the Ministry by Carleton University [8] has resulted in a computer program to predict the depth and time pattern of frost penetration beneath pavement structures.

The computer simulation program involves the creation of a numerical, finite difference, heat transfer model of the soil, and the undertaking of a stepwise digital computation in which actual weather data is used. Computer simulation not only provides detailed predictions of frost depth and ground temperature conditions, but is able to predict complex and important future events.

8.1/ Computer Model

The model as described by Smith [8] is based on the strategy developed by Outcalt (1972, 1975) [10]. The computer program simulates the ground temperature and surface energy regimes for paved surfaces using inputs of synoptic weather data. A simplified flowchart is shown in Figure 15. The main structural elements of the program are as follows.

(a) Inputs of weather data (solar radiation, cloud cover, air temperature, wind speed and atmospheric pressure), together with certain highway characteristics (albedo, aerodynamic roughness) are used to solve the surface energy balance. In general, this is written as:

$$RN = H + S\ (+\ LE)$$

where:

RN = the net radiation balance of incoming and outgoing radiation
H = convection
S = the heat conduction into or out of the ground
LE = the evaporation (assumed to be zero for a highway surface).

(b) The purpose of solving this energy balance equation is to derive the prevailing surface temperature, which is not routinely available anywhere. To do this, the components of the energy balance equation are each expressed as functions of surface (highway) temperature and iterative solution of this equation leads to the equilibrium surface temperature.

(c) The equilibrium surface temperature is then used as the boundary condition, together with the ground thermal properties, to predict the ground temperature pattern via the heat conduction equation.

An implicit finite difference solution is used (Carnahan, Luther and Wilkes, [11]) which accommodates complex stratigraphy (including frozen and unfrozen states) and freeze/thaw energetics. No account is taken of moisture migration during freezing.

The solution is one-dimensional and thus ignores the finite width of the highway. However, since the depth of frost penetration is considerably less than highway width (approximately 1:10), this is not a serious limitation. Predictions may be viewed as being at the highway centreline.

The program is organized to produce output on a daily basis, using either input of actual daily weather data or daily data interpolated from the input of monthly data. The latter is adequate for the general prediction of frost penetration, but if accurate prediction of the timing of freezing and thawing is required, then it is necessary to use actual daily data. Ground thermal properties may be input directly (where available) or calculated in a subroutine on the basis of physical information supplied by the user. This option is specified separately for each soil layer (up to a maximum of nine layers). The initial ground temperature profile may actually be specified where data are available or simply preset to arbitrary values, although the former is preferred. The bottom grid is necessarily kept at a fixed temperature in the finite difference calculations. A depth of 6 m is generally suitable for this.

8.2/ Input Information

The program uses information which, in most cases, is widely available. Soil thermal properties are either input directly or may be calculated by a subroutine. If limited soils information is available, "typical" values may be used.

An initial soil temperature profile must be provided. The data may be direct results from thermocouples or thermistors or, if no information on soil temperature is available, an arbitrary temperature may be set based on the mean annual air temperature at the nearest meteorological station.

Climatological data comprises a large part of the input needed for the program. Records published by the Atmospheric Environment Services provide all of the information for simulations. The type of weather data required is solar radiation, cloud cover, cloud type, air temperature, wind velocity and atmospheric pressure. In most cases, it will be necessary to use more than one station to assemble all the data required.

8.3/ Computer Prediction versus Measured Frost Depth Penetration

Model predictive capabilities were tested on two specially-instrumented sites in the Ottawa area. The sites were instrumented for the measurement of ground water and temperature in a previous study [12]. Soil samples were available from the sites, one of which was predominantly a clay subgrade and the other mainly sand. Weekly measurements of temperature and ground water provided the data on which the program is based. The observed and simulated frost lines (Smith 1976 [8]) are shown in Figure 16 for one of the winters during which measurements were taken.

Although the prediction model is not perfect, the pattern of frost penetration and soil isotherms is realistically produced at both sites. The worst error encountered in predicting the maximum frost penetration was about 10% (15 cm). According to Smith, this disagreement may be due, in part, to the fact that predictions were based on mean daily temperatures, whereas the measurements were all taken in the afternoon when the surface layer would be warmer. Even though the prediction model used very accurate soils information, part of the error may also be due to assigning the soil moisture conditions at the two test sites.

Part of the testing for the model included comparing the results from the frost indicator study with the computer prediction. Four sites were selected from the Ontario network [13] and the necessary weather data was obtained from local weather stations. As only minimal soils information was available, average values were derived from the literature and used in the program.

The observed and simulated frost lines for the four sites are shown in Figures 17 and 18. Agreement is reasonable with only 1% error in the predicted frost depth for Carp, 13% for Millhaven, 15% for Guelph, and 24% for New Liskeard. Although the available soils information is limited, the prediction model correlates reasonably well with the frost depth indicators. It is highly desirable to input accurate soils data; nonetheless, the predictive capabilities of the model are still good even if this knowledge is not available to the designer.

8.4/ Insulating Layers

The potential usefulness of a simulation approach is best illustrated by analysis of the efficiency of different insulation materials in reducing frost penetration. The research engineer has an experimental tool available to fully study the effects of an insulating material in a pavement structure before a fullscale field trail is built. Through this method, effects such as layer thickness, placement in the structure, and efficiency under varying climatic conditions may be studied so that engineers can formulate the best design strategies based on the results.

Figure 19 illustrated a design produced for the Raith Swamp on Highway 17. The effect of an 8 cm layer of extruded polystyrene insulation introduced into the pavement structure is clearly visible as the frost penetration is reduced from 1.84 m without insulation to 0.98 m width with the insulated layer.

If the computer simulation gives favourable results, a research site may be designed and built with confidence that the insulating material will perform as expected. This process would greatly reduce the number of field trials normally needed to determine the best and most economical use of a new insulating material.

The experiment at the Raith Swamp was constructed in 1977. Actual measurements now reveal that the design computer simulation was fairly accurate. However, when the computer simulation was rerun using actual weather and soils data, the results did not agree with those found with any insulating layer. Disagreement was more marked with the thicker insulating layers. The simulation did accurately predict penetration in the uninsulated control section. Further work is needed to establish suitable constants for insulating materials for use in this computer simulation program.

The simulation program did agree with measurements in the Noelville experiment with bark as a frost retarding layer, as is shown in Figure 20.

8.5/ Discussion

Because soil conditions are such an important factor in determining the frost penetration in pavement structures, estimates based on calculations rather than the relationships shown in Figure 9 are more accurate. The graph in Figure 9 is useful, however, for a knowledge of the site air freezing index is required and these indices can be easily obtained from the maps for any location in Ontario.

In trying to predict frost penetration for an area in which construction of a highway is projected, it must be remembered that the depth of frost penetration at any particular location will vary from year to year in relation to the extent and duration of the freezing temperatures experienced. In addition, the soil properties and moisture content will vary throughout any stretch of highway and, thus, the depth of frost penetration will also vary. The advantage of computer simulation is that it is able to predict complex future events without actually having to experience them in a field situation.

The program makes possible more accurate time temperature profiles for use in evaluation of the optimum designs for new pavement structures using experimental materials such as tree bark or sulphur foam. The program could also be used to design for the worst possible situation.

The simulation program requires considerably more data and time to be able to predict frost penetration; for specific project applications, however, the program can be a major tool for the pavement design engineer.

9/ SPRING THAW

Accurate knowledge of the pavement structure condition during the spring thaw would aid Ministry and municipal maintenance engineers in deciding when to erect signs for the half load legislation.

9.1/ Measurements

Ontario's network of frost depth indicators has been used to determine when half load restrictions should apply during spring thaw. The indicators are very accurate when surrounding soil temperatures are below the freezing point of 0°C. As the indicators contain only a very small column of liquid, they tend to thaw much more quickly than the mass of material around them. As a result, the indicator shows a thawed condition, while the pavement structure may still be solidly frozen. The gauges do, however, give a good indication of when the structure is in a thawing condition and may, depending on the weather, rapidly deteriorate to a much weakened state.

9.2/ Computer Simulation of the Spring Thaw

Even though the computer simulation prediction for total frost penetration is reasonably accurate, the same does not apply for the spring thaw prediction. The timing for the thaw below 0.3 m as well as the date of completion can be predicted fairly accurately. The condition of the top 0.2 m to 0.3 m, however, cannot be predicted with certainty as the fluctuating freeze thaw conditions will depend on the day-to-day weather. If the pattern of thaw is required with some accuracy, it is recommended that the monthly weather data normally used be replaced with daily data after March 15.

10/ CONCLUSIONS

This report summarizes the results of five years of observations of frost penetration in pavement structures taken across Ontario. The following conclusions have been made.

(a) The modified Swedish-type frost depth indicator is an inexpensive and reliable instrument for measuring frost penetration. The instruments are not accurate in measuring the spring thaw but only indicate when the pavement structure is in a thawing condition.

(b) The relationship between air freezing index and frost penetration was found to be:

$$P = -0.328 + 0.0578 \ \ F$$

where:

P = frost penetration in metres
F = freezing index in degree-days Celsius

(c) Frost penetration at two nearby points may vary significantly depending on the soil and moisture conditions present at the site. However, for design purposes an additional 0.3 m would cover this variation (80% confidence level).

(d) An up-to-date metric freezing index map of Ontario is presented that was based on data from 224 climatological stations in Ontario. The mean freezing index from the map is within 300 degree-days of actual.

(e) A computer program has been developed that is capable of predicting the depth and time pattern of frost penetration beneath pavement structures. The program will enable research and design engineers to fully evaluate problems such as frost heave treatments or new insulating materials which are now being experimented with in highway construction.

(f) The computer simulation program should be used for specific project applications. The relationship based on air freezing index can be used where only a reasonable estimate of frost penetration is needed.

11/ ACKNOWLEDGEMENTS

The authors gratefully acknowledge the help and co-operation of the District Patrolmen who took the weekly gauge readings.

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Table 1/ Frost Depth Indicator Location Details

Region	Dist. No.	No.	Location	Pavement Thickness (mm)	Soils Log Granular Depth (mm)	Subgrade Material
London	1	1-1	Harrow	83	460	Fine Sand
	1	1-2	Chatham	83	610	Fine Sand & Silt
	1	1-3	Port Franks	83	460	Sand
	2	2-1	Wallacetown	83	460	Silty Sand
	2	2-2	Elginfield	83	460	Clay
	2	2-3	Ayr	83	460	Sand & Fine Gravel
	2	2-4	Langton	83	460	Sand
	3	3-1	Arthur	76	990	Sand
	3	3-2	Wingham	83	610	Clay, Loam
	3	3-3	Guelph	100	460	Sand & Gravel
	3	3-4	Clinton			
	3	3-5	Fordwich			
	3	3-6	Conn			
	5	5-1	20 S. Barrie	76	710	Sand
	5	5-2	Cookstown			
	5	5-3	Clavering	83	460	Sandy Loam
	5	5-4	Elmvale			
Toronto	6	T36	Brampton	140	610	Clay
	4	T6	Brantford	140	230	Gravelly Sand
	4	4-1	Welland	76		Clay
	7	7-1	Bowmanville	230*		Clay
Kingston	8	8-1	Millhaven	102	380	Clay
	9	9-1	Iroquois	76	740	Clay
	9	9-2	Perth	102	660	Clay
	9	9-3	Carp	76		Sand
	9	9-4	Morrisburg	76		
	10	10-1	Bancroft	76	380	Gravelly Silt
	10	10-2	Whitney	76	230	Sand
	10	10-3	Bancroft	76		Sand
North Bay	11	11-1	Bracebridge	76	205	Sand
	11	11-2	Dunchurch	102	410	Sand
	13	13-1	Sunridge	102	180	Sand
	13	13-2	Deep River	102	460	Sand
	14	14-2	New Liskeard	180	690	Clay
	14	14-2	Cobalt	102	660	Sand
	14	14-3	Matheson	102	76	Sand
	14	14-4	Timmins			Clay
	17	17-1	Britt		205	Sand
	17	17-2	Little Current	51	380	Sand
	17	17-3	Cartier	25	1090	Sand
Thunder Bay	16	16-1	Cochrane	102	380	Silty Clay
	16	16-2	Nagagami	152	510	Sand
	16	16-3	Shekak River	140	485	Sand
	16	16-4	Val Gagne	76	915	Clay
	16	16-5	Iroquois Falls	76		Clay
	18	18-1	Wawa	65	725	Sand
	18	18-2	Marathon	76	150	Silty Clay
	18	18-3	Sault Ste. Marie	115	950	Clay
	19	19-1	Thunder Bay	51	510	Clay
	19	19-2	Nipigon	102	1170	Silty Clay
	19	19-3	15 Mi. N. Nipigon	65	850	Sand
	19	19-4	10 Mi. E. Nipigon	140		Heavy Clay
	19	19-5	Thunder Bay	280	760	Clay
	19	19-6	31 Mi. W. Atikokan	130	1080	Light Clay
	20	20-1	Rainy River	40	570	Silty Clay
	20	20-2	Ear Falls	65	420	Sand
	20	20-3	Dryden	115	800	Silty Clay
	20	20-4	Dryden	130	710	Sand
	20	20-5	Sioux Lookout	65	395	Silty Clay
	20	20-6	Sioux Lookout	65	420	Sand
	20	20-7	Red Lake Road	N.D.		
	20	20-8	30 Mi. E. Kenora	130		Sand & Gravel

Table 2/ Ontario Frost Penetration Results 1970-75

No.	Degree Days Celsius (From Map)	Predicted Frost Penetration (m)	1970-71		1971-72		1972-73		1973-74		1974-75	
			Degree Days Celsius	Maximum Frost Penetration (m)	Degree Days Celsius	Maximum Frost Penetration (m)	Degree Days Celsius	Maximum Frost Penetration (m)	Degree Days Celsius	Maximum Frost Penetration (m)	Degree Days Celsius	Maximum Frost Penetration (m)
1-1	300	0.62	Indicator		400	0.71	190	0.53	300	0.61		N.D.
1-2	300	0.62	Installed			0.84		0.74		0.79	170	0.56
1-3	450	0.88	Summer			0.89		0.48		N.D.		0.58
2-1	400	0.78	1971			0.81		0.51		0.86		0.43
2-2	500	0.93				1.05	Damaged		520	0.94	415	0.79
2-3	600	1.06				1.22		1.19		1.22		1.02
2-4	450	0.88				0.91		0.78		0.81		0.61
3-1	900	1.36		1.46	830	1.36	840	1.27	750	1.27	706	1.19
3-2	600	1.06				0.94		0.94		0.94		0.74
3-3	600	1.06				1.57		1.24	705	1.45	650	1.22
3-4	550	1.00			Installed			0.91		0.98	Cancelled	
3-5	700	1.16			Fall			1.51		1.56	Cancelled	
3-6	900	1.36			1972		640	1.42	750	1.50	Cancelled	
5-1	700	1.16		1.51		1.36	Cancelled		Cancelled		Cancelled	
5-2	700	1.16				1.04		0.98		1.07		0.91
5-3	600	1.06			580	1.07	420	0.99	530	1.02	400	0.83
5-4	700	1.16			Installed Fall 1972			1.04		1.16		1.07
T36	500	0.93	720	1.32	670	1.12	480	0.99	580	1.02	Cancelled	
T6	450	0.88		1.40		1.27	Cancelled		Cancelled		Cancelled	
4-1	400	0.78							380	0.71	175	0.64
7-1	500	0.93								1.14		1.14
8-1	650	1.12	880	1.24	No Measurements		855	0.94	860	0.94		0.81
9-1	700	1.16		1.46	Indicator Damaged		Cancelled		Cancelled		Cancelled	
9-2	900	1.36	Indicator Damaged			1.29		1.22		1.07		N.D
9-3	1000	1.45			1110	1.35	870	1.27	930	1.17	770	1.14
9-4	700	1.16			Installed Fall 1972			1.12	Damaged			1.02
10-1	1000	1.45		1.07	1060	1.40	790	1.09	945	1.36		N.D.
10-2	1100	1.53	Indicator Damaged			1.85		1.98		1.90		1.88
					Full Depth of Ind.		New Indicator					
10-3	1000	1.45	New Indicator		1060	1.68	790	1.85	945	1.59		N.D.
11-1	900	1.36		1.90		1.35		1.14	920	1.27		1.24
11-2	1000	1.45	Indicator Damaged			1.54		1.21		1.25		1.27
13-1	1000	1.45	1335	1.61	1390	1.47	1050	1.31	1330	1.38	1120	1.27
13-2	1100	1.53		1.90		1.93		1.74	1090	1.78		1.71
14-1	1400	1.80		2.22	1670	2.18	1360	2.03	1720	2.01	1500	1.79
14-2	1400	1.80		2.36	1670	2.36	1360	2.23	1720	2.34	1500	2.06
14-3	1800	2.10		1.98		2.06		2.01		2.07		1.88
14-4	1750	2.08			Installed Fall 1972			2.06		2.03		2.10
17-1	1050	1.50		1.62		1.55		1.50		1.47		1.42
17-2	1050	1.50		1.45		1.32		1.27		1.42		N.D.
17-3	1400	1.80		1.85	Ind. Paved Over					1.88		1.70
16-1	1850	2.15	2060	2.37	2170	2.48	1800	2.48	2200	2.49		N.D.
16-2	2050	2.27		2.61	2440	2.76	2000	2.82	2390	2.73		N.D.
16-3	2050	2.27			2440	2.56	2000	2.39	2390	2.51		N.D.
16-4	1800	2.10			Installed Fall 1972			2.38		2.49	1660	2.31
16-5	1800	2.10							2100	2.28		N.D
18-1	1400	1.80		2.06		2.27		2.01		2.26		N.D
18-2	1200	1.64		2.06	Indicator Damaged		Cancelled		Cancelled		Cancelled	
18-3	1000	1.46		1.38	1110	2.17	780	1.37	Damaged		Cancelled	
19-1	1350	1.78	1560	1.80	1780	1.70	1280	1.50	1580	1.80	1240	1.38
19-2	1500	1.88		2.01		2.49		2.18		2.16	Cancelled	
19-3	1800	1.98		2.26		2.61		2.16		2.44	Cancelled	
19-4	1500	1.88				2.17		1.83		N.D.	Damaged	
19-5	1350	1.77			1780	1.90	1280	1.73	1580	1.80	Damaged	
19-6	1800	1.98				2.39		2.01		2.16	Cancelled	
20-1	1700	2.04		2.08		2.11		1.83		1.98		1.78
			Full Depth of Ind.		Full Depth of Ind.							
20-2	1900	2.18		1.90		1.98		1.88	2140	1.89	1880	1.76
20-3	1850	2.15	1920	2.09	2060	2.03	1610	2.03	2055	2.01	1850	1.90
20-4	1850	2.15	1920	1.96	2080	2.06	1610	1.96	2055	2.06	1850	2.46
20-5	1900	2.18		1.98	Indicator Damaged		Damaged		2210	1.85		N.D.
20-6	1900	2.18		2.64		2.83		2.56	2210	2.83	1755	2.51
20-7	1850	2.15				2.39		2.18		N.D.		2.18
20-8	1750	2.08			Installed 1972			2.03		2.16		N.D.

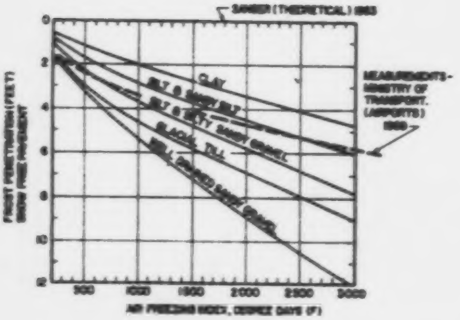


Figure 1/ Frost Penetration Curves Based on Theoretical Calculations

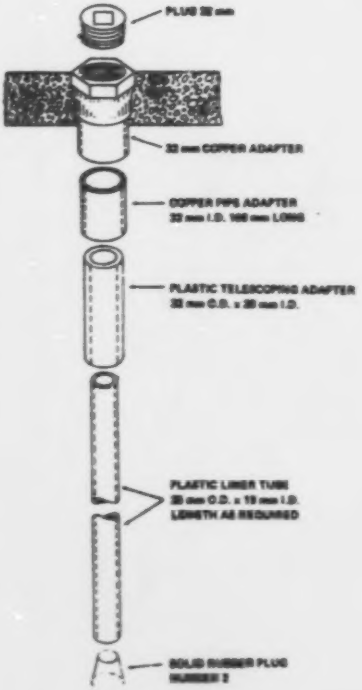

PLUG 32 mm
32 mm COPPER ADAPTER
COPPER PIPE ADAPTER 32 mm I.D. 100 mm LONG
PLASTIC TELESCOPING ADAPTER 32 mm O.D. x 26 mm I.D.
PLASTIC LINER TUBE 25 mm O.D. x 19 mm I.D. LENGTH AS REQUIRED
SOLID RUBBER PLUG NUMBER 2

Figure 2/ Gandahl Type Frost Depth Indicator Casing Assembly (MTC Design)

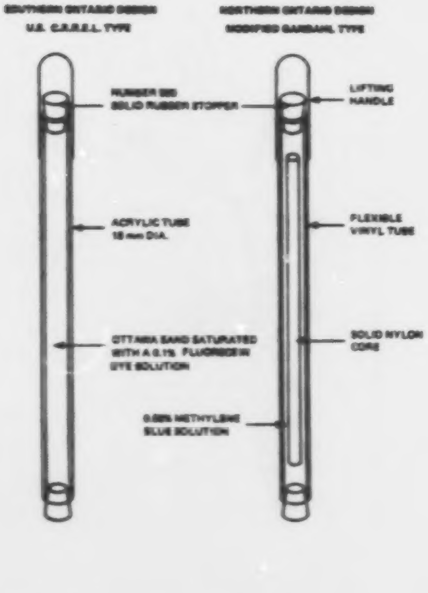

SOUTHERN ONTARIO DESIGN
U.S. C.R.R.E.L. TYPE
NORTHERN ONTARIO DESIGN
MODIFIED GANDAHL TYPE
NUMBER 00 SOLID RUBBER STOPPER
LIFTING HANDLE
ACRYLIC TUBE 18 mm DIA.
FLEXIBLE VINYL TUBE
OTTAWA SAND SATURATED WITH A 0.1% FLUORESCEIN DYE SOLUTION
SOLID NYLON CORE
0.02% METHYLENE BLUE SOLUTION

Figure 3/ Gandahl Type Frost Depth Indicator Assemblies (MTC Design)

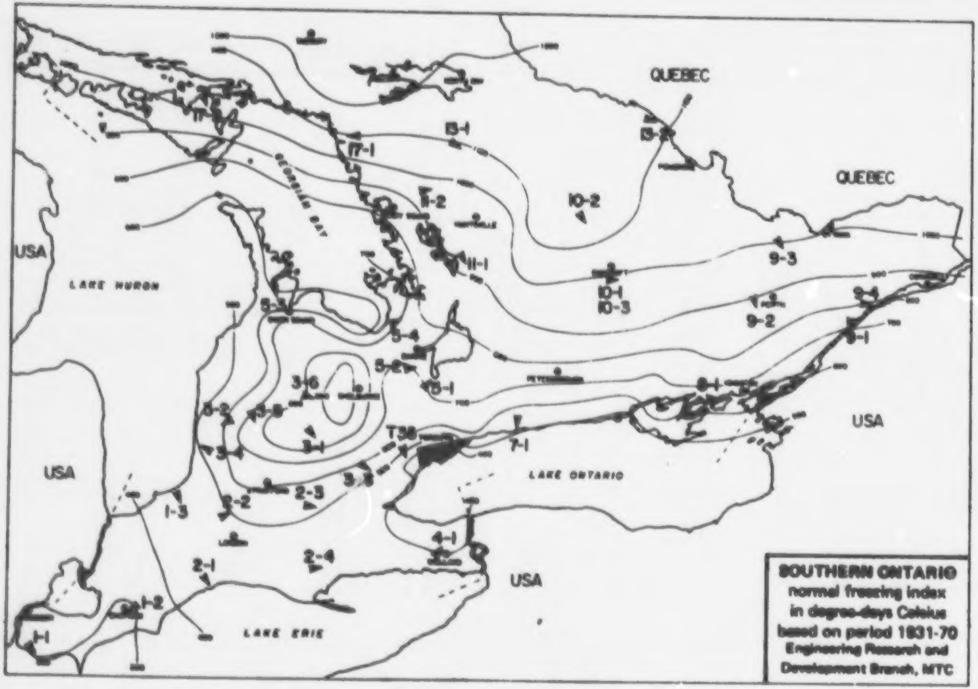

QUEBEC
QUEBEC
USA
USA
USA
USA
LAKE HURON
GEORGIAN BAY
LAKE ONTARIO
LAKE ERIE
13-1
13-2
17-1
11-2
10-2
11-1
9-3
10-1
10-3
9-2
9-4
9-1
8-1
5-4
5-2
15-1
3-6
5-2
3-8
3-1
T38
7-1
5-4
3-3
2-3
2-2
1-3
4-1
2-4
2-1
1-2
1-1
SOUTHERN ONTARIO
normal freezing index
in degree-days Celsius
based on period 1931-70
Engineering Research and
Development Branch, MTC

Figure 4/ Frost Depth Indicator Locations 1974 - 1975, Southern Ontario

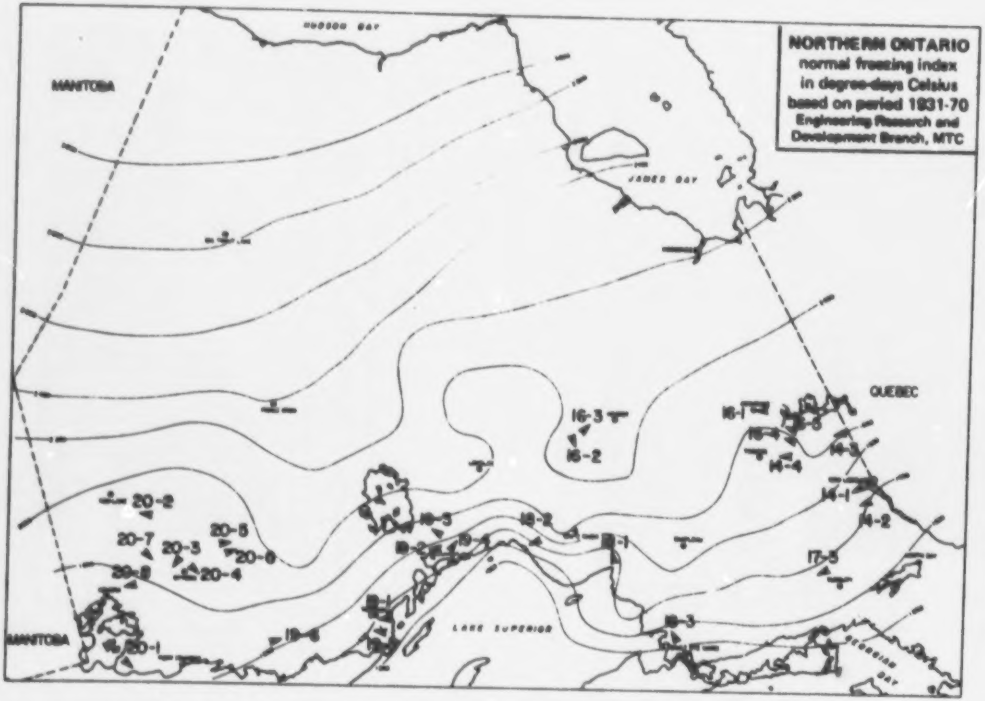
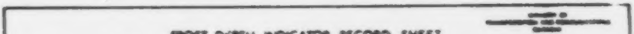

NORTHERN ONTARIO
normal freezing index
in degree-days Celsius
based on period 1931-70
Engineering Research and
Development Branch, MTC
HUDSON BAY
MANITOBA
JAMES BAY
QUEBEC
LAKE SUPERIOR
GEORGIAN BAY
MANITOBA
16-3
16-2
16-1
16-4
14-3
14-4
14-1
14-2
17-5
18-2
18-3
18-1
20-2
20-7
20-3
20-5
20-6
20-4
20-1
19-8
19-3

Figure 5/ Frost Depth Indicator Locations 1974 - 1975, Northern Ontario



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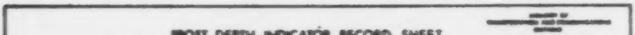


Figure 7/ Example Data Sheet from Northern Region of Ontario

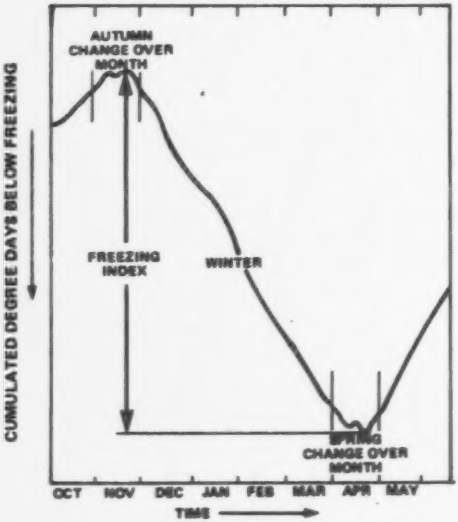


Figure 8/ Calculation of Freezing Index

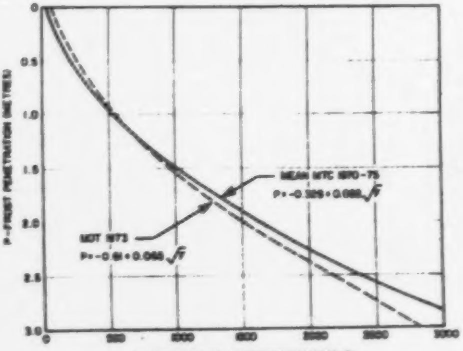


Figure 9/ Frost Penetration in Ontario 1970-1975

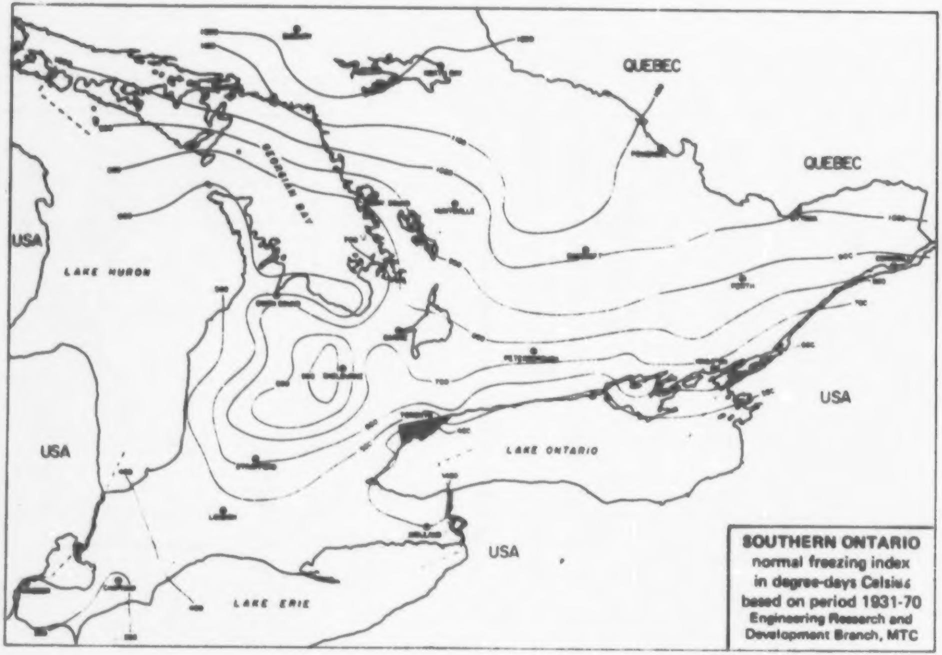
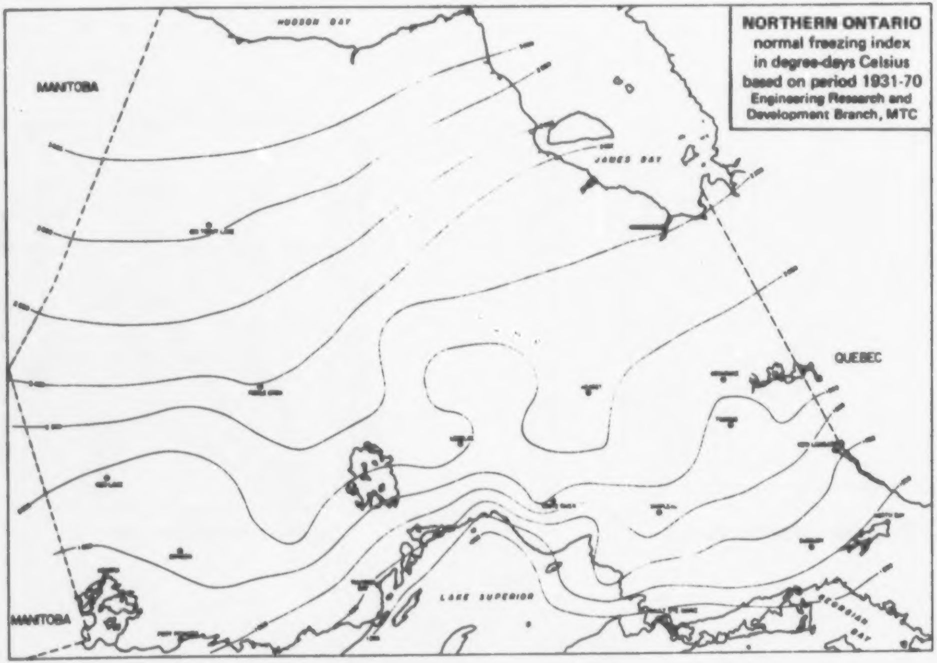

QUEBEC
QUEBEC
USA
USA
USA
USA
LAKE HURON
GEORGIAN BAY
LAKE ONTARIO
LAKE ERIE
SOUTHERN ONTARIO
normal freezing index
in degree-days Celsius
based on period 1931-70
Engineering Research and
Development Branch, MTC

Figure 10/ Freezing Index Map of Southern Ontario


NORTHERN ONTARIO
normal freezing index
in degree-days Celsius
based on period 1931-70
Engineering Research and
Development Branch, MTC
HUDSON BAY
JAMES BAY
MANITOBA
QUEBEC
MANITOBA
LAKE SUPERIOR
GEORGIAN BAY

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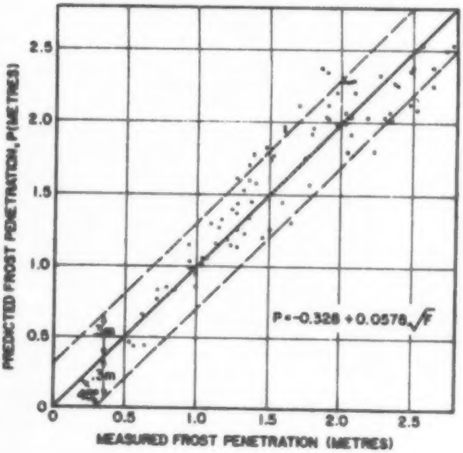
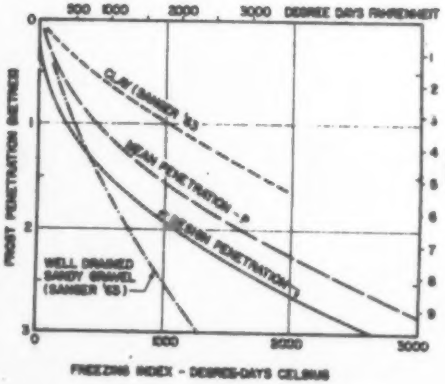


Figure 12/ Observed and Predicted Frost Penetration



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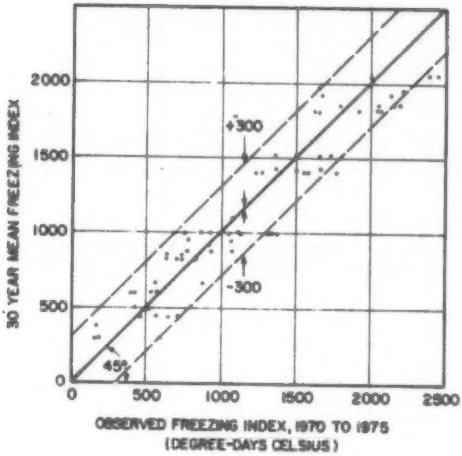
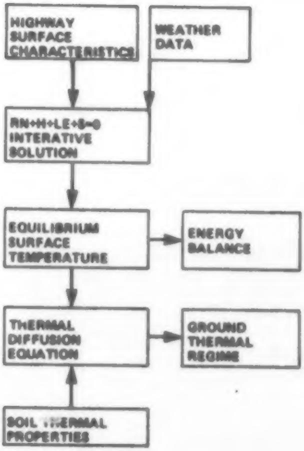


Figure 14/ Observed and Predicted Freezing Index


HIGHWAY SURFACE CHARACTERISTICS
WEATHER DATA
RN+H+LE+S=0 INTERATIVE SOLUTION
EQUILIBRIUM SURFACE TEMPERATURE
ENERGY BALANCE
THERMAL DIFFUSION EQUATION
GROUND THERMAL REGIME
SOIL THERMAL PROPERTIES

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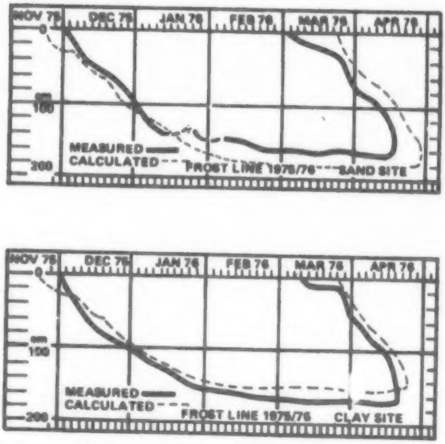

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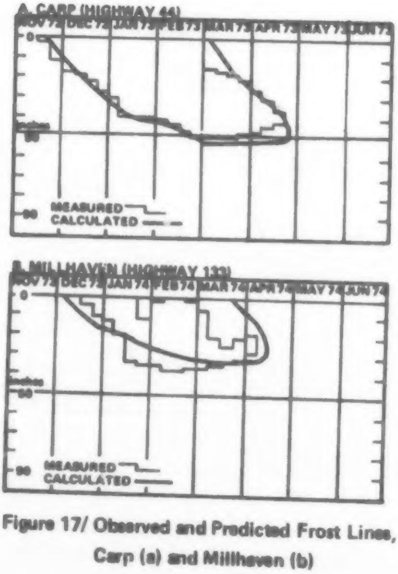

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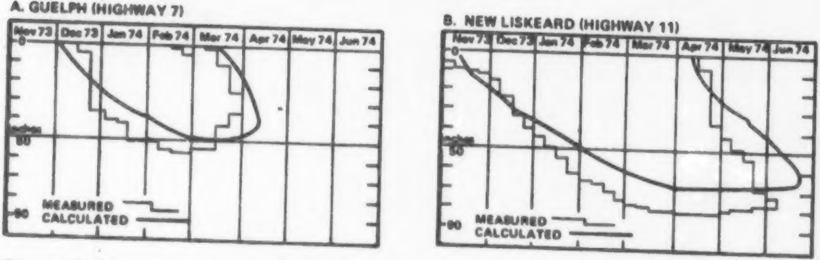

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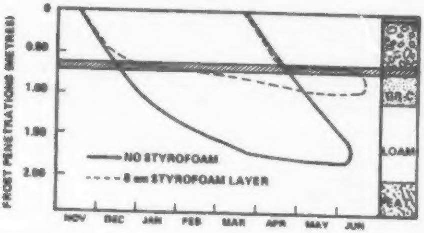

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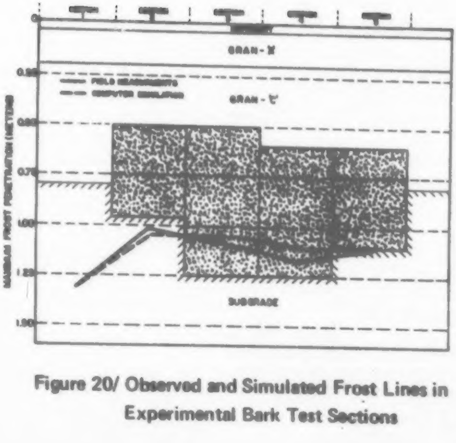

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Session 3

Frost Heave Preventing Measures in Road Pavements

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ABSTRACT

In snowy and cold areas, road pavements, concrete tunnels and retaining walls are greatly affected by freezing and thawing of the earth. Expecially in Hokkaido, and in warmer mountainous areas, frost heave in road pavements causes serious consequences.

The frost heaving occurs when ice crystals grow in the freezing direction and, thus, cause the earth surface to heave. The phenomenon causes damage in two forms: one is due to the frost heaving itself, while the other arises from a decrease in the bearing capacity of the subgrade and subbase in pavements during the spring period. A large number of heavy vehicles cause partial rutting and alligator cracking in asphaltic pavements, while drainage is obstructed by the frozen underlayers in the spring.

The structure of asphaltic pavements is designed with consideration of the number of heavy vehicles, the design CBR of subgrade soil and the depth of frost penetration in pavements in cold regions. In order to protect pavements from frost-heaving damages, the displacement, heat insulation, soil stabilization, and the installation of impermeable layers have been selected as countermeasures, both experimentally and for practical purposes.

For reasons of economy, performance and experience, the replacement method is now generally used for road pavements in Japan. This method consists of replacing frost-susceptible soil with coarse materials to approximate 70% of the theoretical maximum frozen ground in winter.

The Bibi Test Road has been established on National Highway No. 36 in the Tomakomai district, Hokkaido, for the purpose of conducting systematic studies for countermeasures against

frost heave damages in the cold areas and to make contributions to theoretical analysis of asphaltic pavements.

This paper will describe the case history of the design method to avoid frost-heaving damages on roads in Hokkaido, and will show a frost-preventing method based on the results of the Bibi Test Road and other field tests with the use of insulation in the pavement structures.

FROST HEAVE PREVENTING MEASURES IN ROAD PAVEMENTS

by
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Hokkaido Development Bureau

Introduction

In cold regions, road pavements, concrete tunnels and retaining walls are greatly affected by frost heave in the ground. Road damages caused by frost heave are often seen not only in Hokkaido and Northern Honshu, but also in mountainous areas in the warmer parts of Japan.(1)

Frost heave is defined as a phenomenon where ice lenses in the subgrade soil grow in the freezing direction and thus cause the ground surface to rise. Frost damage to pavement generally occurs when frost heave reaches a maximum along the center line, causing large longitudinal cracks. This kind of frost damage greatly depends upon the degree of snow removal.

Another kind of frost damage in pavement is due to a decrease in the bearing capacity of the subgrade and subbase course in spring.(2) In order to protect pavements from frost heave damages, displacement, chemical treatment, heat insulation, and water interception methods have been used as countermeasures, both experimentally and practically. For reasons of economy, performance, and reliability, the replacement method is now generally used for road pavements in Japan. This method consists of replacing frost-susceptible soil with coarse-grained materials down to about 70 percent of the maximum depth of the frozen ground.(3) Based on extensive experience over many years, this method has been used in the standard design of all classes of roads in cold regions.

When pavement structures are designed and constructed in cold regions, it is necessary to calculate the frost penetration depth, to determine pavement thickness and to evaluate the bearing capacity of the subgrade and subbase course during the thawing period, and the results greatly affect the kind and thickness of the replacement material.(4) It is further necessary to conduct the investigations of the actual frost heave damages in the field. This paper describes problems caused by frost action and the methods of frost protection in road pavement in cold regions in Japan.

1. Frost Damage on Pavements

Frost damages can be grouped into two categories: (a) Damage due to the frost heave itself, as when ice lenses grow in ground soil and cause the ground surface to rise, and (b) damage from the decrease in bearing capacity on the subgrade and subbase soils caused by frozen layers thawing in spring.

1.1. Damage from the Frost Heave

Damage of this type occurs when frost heave causes unevenness in the transverse direction of the road, growing largest along the center line and causing large cracks, as shown in Figure 1. This uneven formation of frost heave greatly depends on the exposure of the road surface, affected by snow removal from the road. At the shoulder of the road the pavement is insulated by snow piles, while the central portion loses the thermal insulation effect of snow due to snow removal and heavy traffic loads. Accordingly the central portion freezes deeply, and shows a greater degree of frost damage than the shoulder portion, thus causing stress-strain destruction of the pavement. This kind of destruction on asphaltic pavements is characterized by its occurrence in the longitudinal cracks of roads mainly at the central joints.

Concrete coverings in tunnels are liable to suffer frost damage when they cover soft rock such as clayey rock and shale. Relatively short tunnels, less than a few hundred meters long often suffer frost damage, since the depressed air penetrates from the openings.(5) Complete snow removal from the shoulder portion of roads results in additional damage from frost heave. Figure 2 shows that snow removal causes the pavement surface to rise in the shoulder portion and to form cracks there in winter.

1.2. Frost Damage Caused by the Decrease in Bearing Capacity

During the spring period ice layers in the subgrade soil begin to thaw mainly from the ground surface in the frozen ground. As a consequence, the flow of thawed water in the ground is arrested by frost layers in the lower parts of the frozen ground and cannot drain into the subsoil. Thus, a considerable amount of thawed water may be trapped in the ground, with the result that saturation of the soil may occur. Since the density of the soil may be small because of the presence of ice layers in the ground, the bearing capacity is very low. When heavy wheels pass over roads in this condition, pavements are damaged greatly and there is the increase of the tensile stress-strain working under the various layers of the pavements. Asphaltic pavements produce alligator cracks on the surface course, as shown in Figure 3.

2. Depth of Frost Penetration and Frost Heaving

Depth of frost penetration and frost heaving in road pavements usually depend on temperature, soil, groundwater and many other factors such as radiation of the sun, kind of pavement, road-surface color and snow on the road. It may be possible to estimate the frost depth from weather observation data and the physical properties of the road component materials, but it is more desirable to obtain the freezing depth by measuring the underground temperatures during February to March when the maximum freezing depth is easily observed.

There are several ways to obtain the frost depth by actual measurements. One way is to use a frost depth gauge with methylene blue; another is to use a resistance thermometer or a thermocouple buried in the ground; and a third way is to observe the underground temperatures and details of the frost action in excavated test pits during winter.

The amount of frost heave is the difference in height of the road surface before and after ground freezing, and it is nearly equal to the total addition from the thicknesses of the ice layers formed in the ground.

2.1. The Method for the Use of a Frost Depth Gauge with Methylene Blue

This method is to confirm the 0 °C isotherm in the ground, not to confirm the temperature distribution in the ground or other aspects of frost action.(3) The gauge devised by Gandal in Sweden, is shown in Figure 4. From time to time during the freezing period the frost gauge tube is pulled out of the inside tube which is buried in the ground. Depth of frost penetration is obtained by measuring the length of tube color which changes from blue to white in freezing layer.

Considering that this frost depth gauge may be moved by the frost action, it is necessary to measure the height of the ground surface before freezing and the top of the inside tube when measuring during the freezing period, in relation to some neighboring standard height immovable in winter.

2.2. The Method Using an Excavated Test Pit

This method is to obtain the vertical distribution of the underground temperatures in the freezing period and to observe other aspects of the frost heave in the ground.(3) It is desirable to make this observation continuously, but in practice the survey is conducted only once in the maximum freezing period.

A test pit is excavated in a road that is always free of snow, in a position where it is the least possible obstacle to traffic. The pit is about 1 meter wide, in the transverse direction of the road, 1.5 meter long, in the longitudinal direction of the road, and is deeper than the frost depth. The aspects of frost heave on the walls in a pit show various patterns of freezing action, as shown in Table 1. The patterns are classified into five groups in the order of increase in frost heave effect.

3. Frost Heaving Within the Frozen Ground

In the determination of the replacement thickness, as a frost damage countermeasure, it is important to confirm the extent to which frost heave affects each layer in a gravel road. A frozen gravel road, when excavated, reveals the development of a majority of the frost crystals right under the gravel layer and the greater part of the frost heave is in the upper subgrade soil with a great decrease in the ice formation towards the lower layers.

Figure 5 shows a relationship, derived by Ifukube's theory based on data from frost heave surveys carried out on gravel roads in Hokkaido in 1952 to 1953, between frost heave and frost penetration in gravel roads.(4) The x axis is the depth expressed as the ratio of any depth to the maximum frost penetration depth in percent and the y axis is the frost heave expressed as the ratio of frost heaving at any depth to total frost heaving at the road surface in percent. From this figure it is clear that replacement of 80 percent of the maximum frozen ground in existing gravel roads will result in practically no frost heaving, since the possible frost heaving after replacement is statistically only 1 percent of the total frost heaving.

4. Depth of Frost Penetration

Since the depth of frost penetration varies a great deal depending on the kind of soils, groundwater, radiation of the sun and amount of snow, it is very difficult to obtain the most accurate depth through only calculation. However, the very convenient Stefan and Neumann Equation is available.

By applying the Berggren Equation(6), a simplified version of the above equation, in the estimation of the frost penetration depth of gravel roads in Japan, Ifukube indicated that the values were in good agreement with the values actually measured.(4) Further, the author provided that this simplified equation could be applied to pavements constructed with non-water-absorptive materials such as insulating boards, asphaltic layers, or cement-concrete in the pavements.(1)

4.1. Depth of Frost Penetration in a Multi-Layered Soil

In order to obtain the frost penetration in a multi-layered soil, as in road pavements, Aldrich shows Berggren's Equation as follows (6):

$$Z = \lambda \sqrt{\frac{172800\ F}{(L/K)_{eff}}}$$

$$\text{In which } (L/K)_{eff} = \frac{2}{X^2} \Big\{ L_1 d_1 \left(\frac{d_1}{2K_1}\right) + L_2 d_2 \left(\frac{d_1}{K_1} + \frac{d_2}{2K_2}\right) + L_3 d_3 \left(\frac{d_1}{K_1} + \frac{d_2}{K_2} + \frac{d_3}{2K_3}\right) + \text{------} \; L_n d_n \left(\frac{d_1}{K_1} + \frac{d_2}{K_2} + \text{----} + \frac{d_n}{2K_n}\right) \Big\}$$

Here:

X: $d_1 + d_2 + --- + d_n$: Expected frost depth (cm)

d_n: Thickness of each layer within the expected frost depth, d_1 is the thickness of the top layer (cm)

K_n: Thermal conductivity of each layer (Cal/cm·S·°C)

L_n: Latent heat of fusion in each layer (Cal/cm^3·°C)

$\lambda = f(\mu, \tau)$: Correction coefficient

$\mu = Q_{wt} \cdot F / L_{wt} \cdot t$: Fusion parameter

$Q_{wt} = (Q_1 d_1 + Q_2 d_2 + --- + Q_n d_n)/X$: The weighted average of the heat capacity, Q_1 means that of the top layer (Cal/cm^3·°C)

$L_{wt} = (L_1 d_1 + L_2 d_2 + --- + L_n d_n)/X$: Latent heat of fusion expressed as a weighted average, L_1 is the latent heat of fusion in the top layer (Cal/cm^3)

In order to apply Berggren's Equation to the frost penetration of pavement structures, it is necessary to obtain the thermal properties for each of the layers composing the pavement. The author indicated the thermal properties of road materials in Hokkaido by actually measuring their average water contents and dry densities in the field, as shown in Table 2.(7) The thermal conductivity of materials composing pavements was obtained by Kersten's Equation.(8)

When the thickness of each pavement layer, materials, freezing period and freezing index are available, it is possible to obtain the fusion parameter μ from Table 2, the thermal ratio τ from Figure 6 and the correction coefficient λ from Figure 7 under an expected frost depth X.

4.2. Application of the Calculation Method for Frost Penetration in Pavement Design

In order to apply the method of obtaining the frost depth through calculation to a pavement design, it is necessary to simplify the calculation method and to show the improved method in the form of specifications.(3,9)

An approximate relationship between the freezing index and the frost penetration is obtained by choosing the thermal properties, water content, dry density, freezing period and freezing index. When it is supposed that the soils composing the subgrade and antifrost layer in pavement structure have the water content of 15 percent and the dry density of 1.8 grams per cubic centimeter, and that the pavement construction is performed with coarse-grained non-frost susceptible materials, then the relationship between depth of frost penetration and freezing index may be shown as in Figure 8.(9)

4.3. Theoretical Maximum Frost Penetration Depth

In the determination of a replacement thickness of road pavements in cold regions, about 80 percent of the maximum freezing depth of existing gravel roads, as suggested by Ifukube, has been considered the standard replacement thickness. This thickness is based on the results of field observations in gravel roads frozen 30 to 50 centimeters deep. However, with the recent disappearance of gravel roads and the large variations in the frost depth, about 80 percent of the standard replacement has shown disadvantages. The author et al. have suggested that the theoretical maximum frost penetration should be set as the standard replaced depth in pavement structures.(7)

This theoretical maximum frost penetration depth is supposed to determine the replaced thickness for frost heave countermeasures. The maximum frost depth of ground composed of non-frost-susceptible coarse-grained materials uses the maximum depth recorded in the past ten years. One way to obtain the theoretical maximum depth of frost penetration of general ground from the temperature observation data is to read, from Figure 8, the depth of frost penetration which corresponds to the maximum freezing index in the past ten years for the planned location.

5. Anti-Frost Construction Methods

The frost heave phenomenon occurs when all factors relating to soil, temperature, and groundwater are affected. This means that the removal of or improvement in one or more factors may lead to prevent or control of the frost action.

There are several construction methods for frost heave prevention: a method replacing frost-susceptible soil with non-frost susceptible materials (replacement method), a method constructing a heat insulating layer in the subgrade for reduction of frost heave damages by preventing the temperature depression of the frost-susceptible soil (heat insulation method), a method changing the nature of the soil by mixing cement or some chemical compound with the frost-susceptible soil and thus decreasing the freezing temperature (chemical treatment method), and a method of constructing a water intercepting layer with vinyl sheets or asphalt buried in the subgrade soil for isolating the frost-susceptible soil from the groundwater (water interception method). Of these, the method replacing the frost-susceptible soil with non-frost susceptible material is considered the most affective, and is now the most widely adopted countermeasure against frost heave.

The heat insulation method is used when the frost depth is large with relatively little traffic, and where good replacement materials are not available. The chemical treatment method is applied in combination with the replacement method. The water interception method is rarely used in Japan, as it is difficult to construct in the field.

5.1. The Determination of the Replacement Thickness

The replaced thickness determined by the replacement method is set so that two forms of frost heave damages are prevented. Destruction of the pavement is caused by the frost heave itself and the decrease in bearing capacity of the subgrade and subbase in the pavement during the thawing period. In order to prevent the damage from the frost heave itself it is theoretically desirable to replace all frost-susceptible soil down to depth of frost penetration with suitable non-frost-susceptible materials.

This method will also prevent pavement destruction caused by the decrease in bearing capacity of subgrade and subbase in spring. However, in Japanese cold regions, it is, economically and experimentally, acceptable to replace frost-susceptible soils with coarse-grained materials down to only about 70 percent of the theoretical maximum frost penetration depth. At present the best guide of replaced thickness is specified from local experiences. Figure 9 shows the general cross section of road pavement structures containing a frost heave preventing layer in Hokkaido.

5.2. The Quality of Fine-Grained Materials Used

Materials used to the replaced depth of a pavement should be non-frost susceptible and have the standard qualities required for use in subbase and antifrost layers. The following specifications are provided for non-frost susceptible fine-grained material for cold regions in Japan(3).

(1) Sand: The amount of the fraction passing a 74 micron

sieve should be less than 6 percent by weight.

(2) Unscreened gravel: Of the total material passing a 4.76 millimeter sieve, the fraction passing a 74 micron sieve should be less than 9 percent by weight.

(3) Crusher-run: Of the total passing a 4.76 millimeter sieve, the amount passing a 74 micron sieve should be less than 15 percent by weight.

(4) Mixed gravel and crusher-run: Of the total material passing a 4.76 millimeter sieve, the amount passing a 74 micron sieve should be less than 12 percent by weight.

(5) Volcanic ash and cinder: Should be coarse-grained, drain well, and have no sign of weathering. The amount passing a 74 micron sieve should be less than 20 percent and its ignition loss less than 4 percent, both by weight. They should also be found non-frost susceptible by a frost heaving test.

5.3 Frost Heave Countermeasures by Other Methods

For the replacement method, which is widely adopted in Japan as a very effective countermeasure against frost heave, there are still some problems to be solved. These include a shortage of non-frost susceptible materials with high quality. It is therefore a matter of urgent concern to develop other methods that can be used in addition to the existing replacement method. As other frost heave countermeasures, there are heat insulation methods, chemical treatment methods, and water interception methods. Of these, the first two are considered to be most promising in Japan.

(1) Heat Insulation Methods

Insulating materials were used initially for frost heave prevention of road pavements in Japan around 1963. Highly effective insulation materials have been developed by high polymer chemistry, and they have now come into use as road insulating materials. The new materials have excellent insulating properties owing to long durability and unchanged quality-volume under traffic loads. Extruded polystyrene boards have been used in Hokkaido for more than 15 years.

Figure 10 shows an example of heat insulation by building a heat insulating layer between the subgrade and subbase course, preventing frost heave in the frost-susceptible soil. Care should be taken, because there is the danger that rolling machinery may break the insulating boards when placing the subbase materials on the heat insulating layer. This can easily be prevented by rolling a 20 to 30 centimeter deep sand or gravel layer on the heat insulating boards. Figure 11 shows a new economical method for heat insulation, in which an existing gravel road is paved with heat insulating material buried only in the shoulder portion.(10)

There is another method using foaming concrete, produced by mixing cement, sand, and foaming beads in a concrete mixer at the construction site. This method is economical as a frost heave countermeasure in sidewalks, because it is suitable for the use of machinery and it can be used in upper layer near the surface of a pavement owing to the high compressive strength gained after curing.(11)

(2) Chemical Treatment Methods

Some chemical additives such as sodium chloride, calcium chloride, lime, and cement, frost-susceptible soil, show a considerable decrease in frost susceptibility. These additives are considered to cause a solidification to soil grains, the depression in the freezing point of the water in pores, an increase in adsorption and a decrease in the water permeability of the soil grains.

It is generally known that cement is most effective in preventing frost heave in sandy soils and lime in silty or clayey soils. Two kinds of lime, slaked lime and caustic lime, are used in this method and the latter is known to be more effective in subgrade soils with high water contents.

6. Concluding Remarks

Research on frost heave damage in road pavements and countermeasures against frost heave in cold regions, have been conducted continuously throughout Japan, and in Hokkaido since 1951 particularly in the field tests. Especially in the field surveys carried out from 1960 at the Bibi Test Road, many attempts have been made to calculate freezing depth, to examine various countermeasures against frost heave damage, and evaluate pavement structures constructed by the replacement method (See Figure 12). The results have given theoretical support to data obtained from experience with actual frost damages. The results in the surveys are considered to be practically useful in Honshu as well as in Hokkaido.

For the replacement method, which has come into the wide use due to its reliability and economy, there are still some problems to be solved. These problems are caused by a shortage of sand and gravel owing to the recent emphasis on conserving seashores and rivers. It is, therefore, necessary to reconsider the use of other methods, though they have not been widely adopted for reasons of economy, and difficulties with the construction. These methods include chemical treatment by the use of cement and lime, heat insulation by placing foaming beads mixed with cement mortar on the roads, and intercepting capillary water by burying vinyl sheets in frost-susceptible soil.

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Table 1 Classification of Frost Pattern in Soils

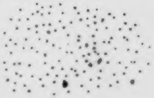
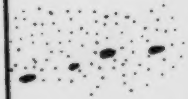
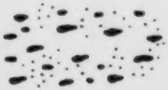
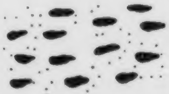
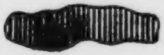
Number	1	2	3	4	5
Classification	Concrete type	Concrete type with fine sirloin type	Fine sirloin type	Sirloin type	Ice filament
Frost pattern					
Explanation	No icelenses	A few fine icelenses	Very fine icelenses	1 - 2 mm icelenses	Well developed ice filament

Table 2 Thermal Properties of Pavement Structural Materials

Name of Materials		Thermal conductivity (cal/cm sec°C)	Volumetric heat capacity (cal/cm^3°C)	Latent heat of fusion (cal/cm^3)
Asphaltic concrete		0.00346	0.448	0
Cement concrete		0.00224	0.480	0
Crushed rock	γ_d = 2.0 g/cm^3 w = 5 %	0.00509	0.415	8.0
Screened gravel	γ_d = 2.0 g/cm^3 w = 7 %	0.00600	0.445	11.2
Sand	γ_d = 1.65 g/cm^3 w = 15 %	0.00470	0.466	19.8
Volcanic ash	γ_d = 0.95 g/cm^3 w = 50 %	0.00275	0.518	38.0
Frost-susceptible soil	γ_d = 1.15 g/cm^3 w = 50	0.00343	0.627	46.0

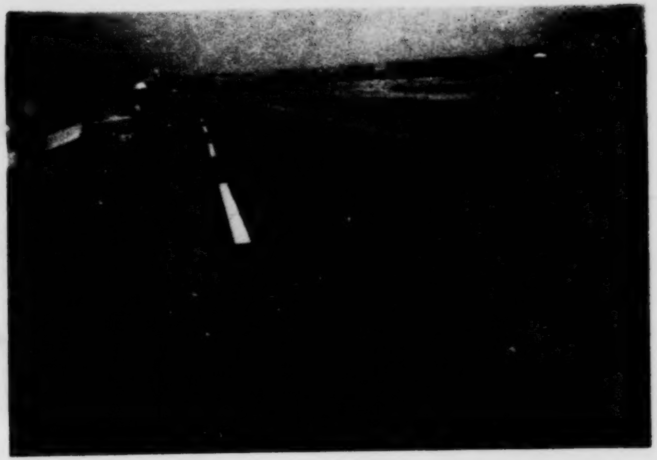


Figure 1 Longitudinal Cracks on a Flexible Pavement Resulting from Frost Heaving of Subgrade Soils

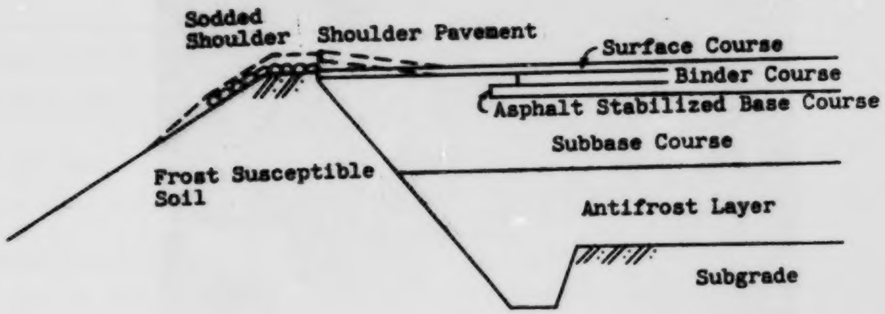

Sodded Shoulder
Shoulder Pavement
Surface Course
Binder Course
Asphalt Stabilized Base Course
Subbase Course
Frost Susceptible Soil
Antifrost Layer
Subgrade

Figure 2 Frost Damage on a Road Shoulder-portion

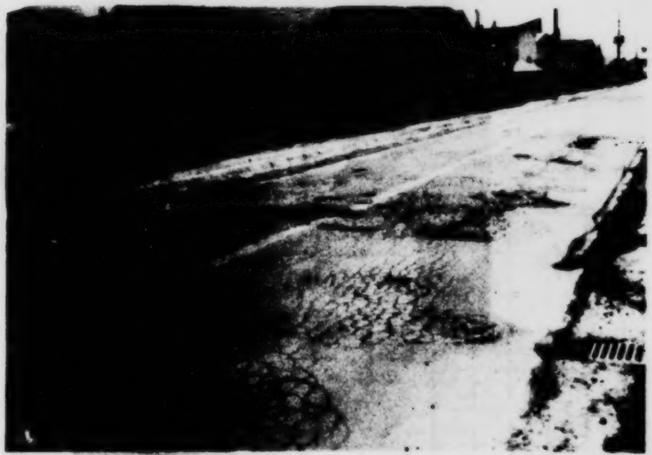


Figure 3 Alligator Cracks Resulting from a Reduction in Bearing Capacity during the Melting Periods

Figure 4 Frost-depth Gauge with Methylene Blue

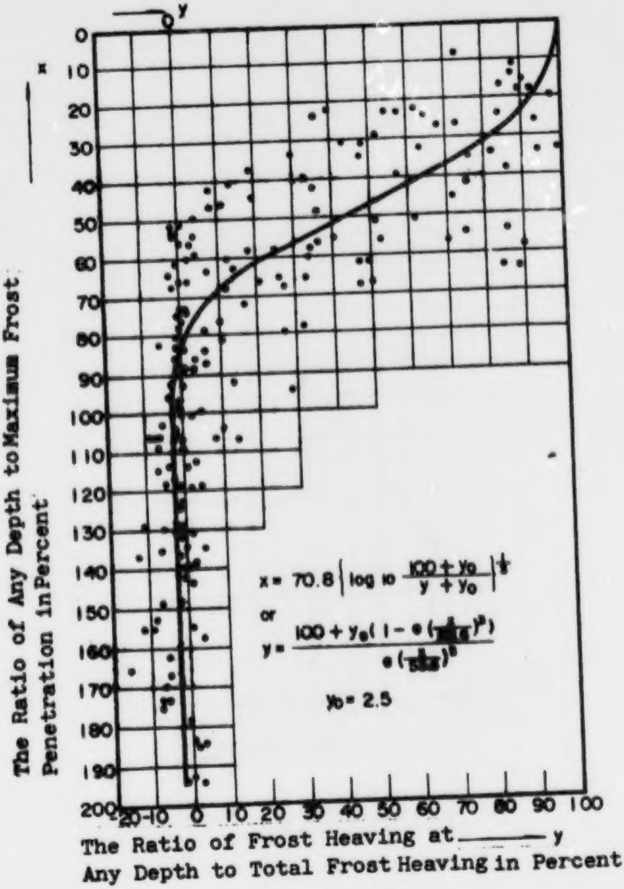


Figure 5 The Influence of Frost Heaving and Penetration on Gravel Roads

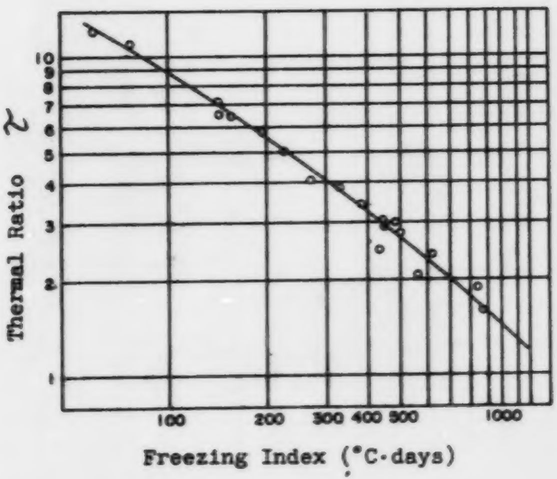


Figure 6 The Relationship between Freezing Index and Thermal Ratio

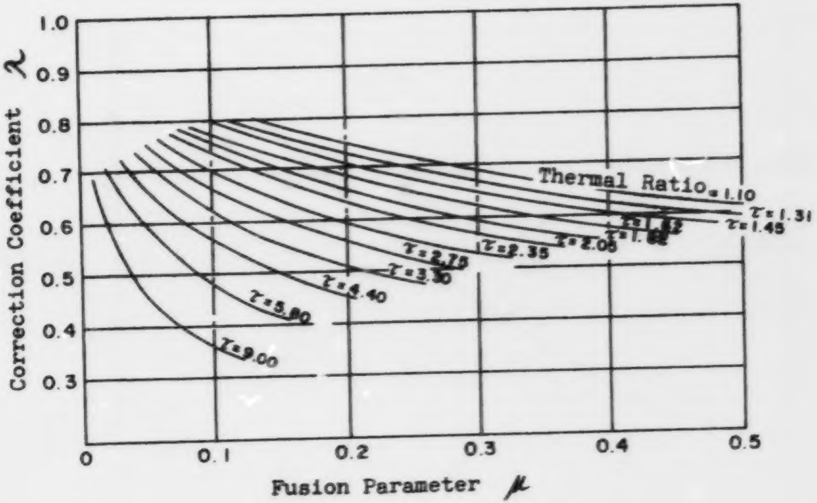


Figure 7 Correction Coefficient in the Modified Berggren Formula

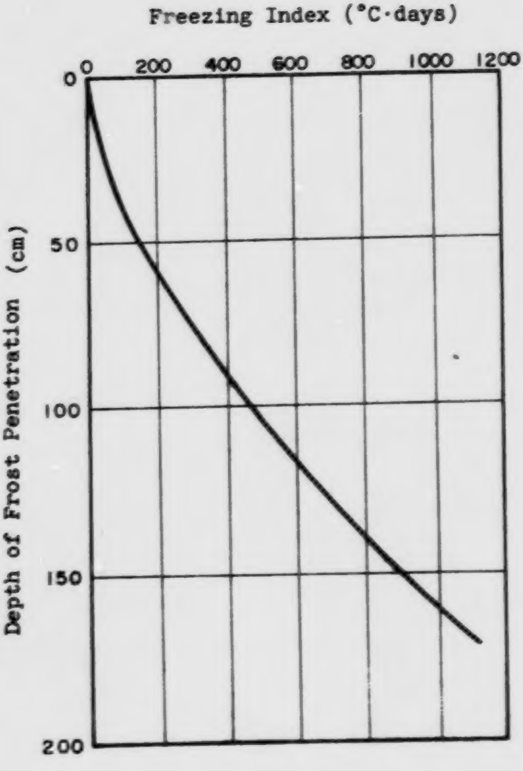
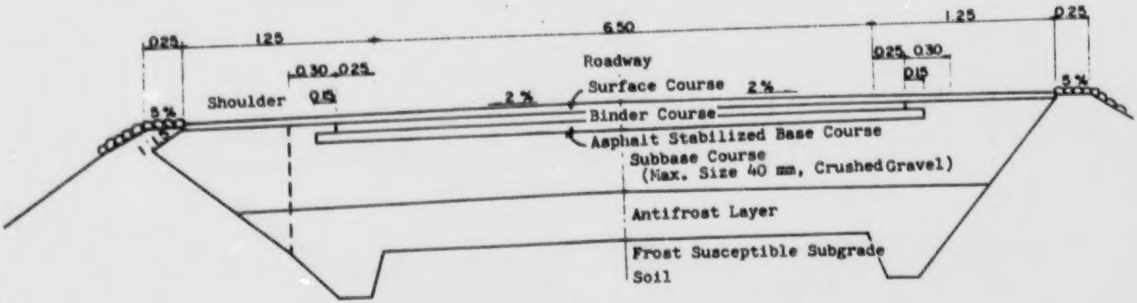


Figure 8 The Relationship between Freezing Index and Frost Penetration in Non-Frost-Granular Grounds


0.25
1.25
6.50
1.25
0.25
0.30 0.25
0.15
0.25 0.30
0.15
Roadway
Shoulder
5 %
5 %
2 %
2 %
1:1.5
Surface Course
Binder Course
Asphalt Stabilized Base Course
Subbase Course
(Max. Size 40 mm, Crushed Gravel)
Antifrost Layer
Frost Susceptible Subgrade
Soil

(Dimension: meter)

Figure 9 General Cross Section of Pavement Structure in Hokkaido

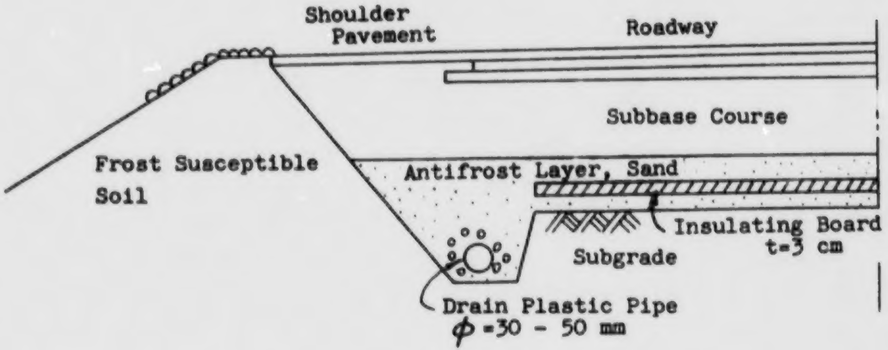

Shoulder
Pavement
Roadway
Subbase Course
Frost Susceptible
Soil
Antifrost Layer, Sand
Insulating Board
t=3 cm
Subgrade
Drain Plastic Pipe
ϕ=30 - 50 mm

Figure 10 An Application of the Insulating Method as Part of the Frost Preventing Materials

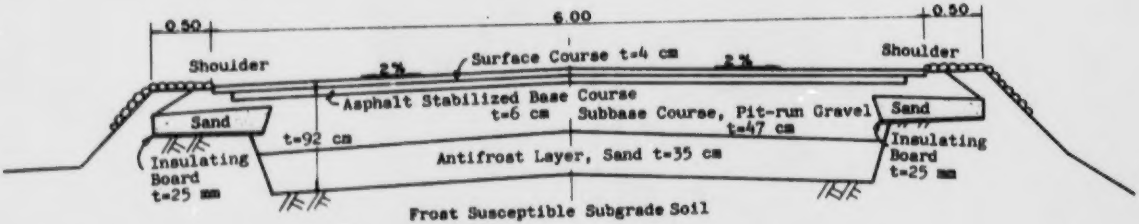
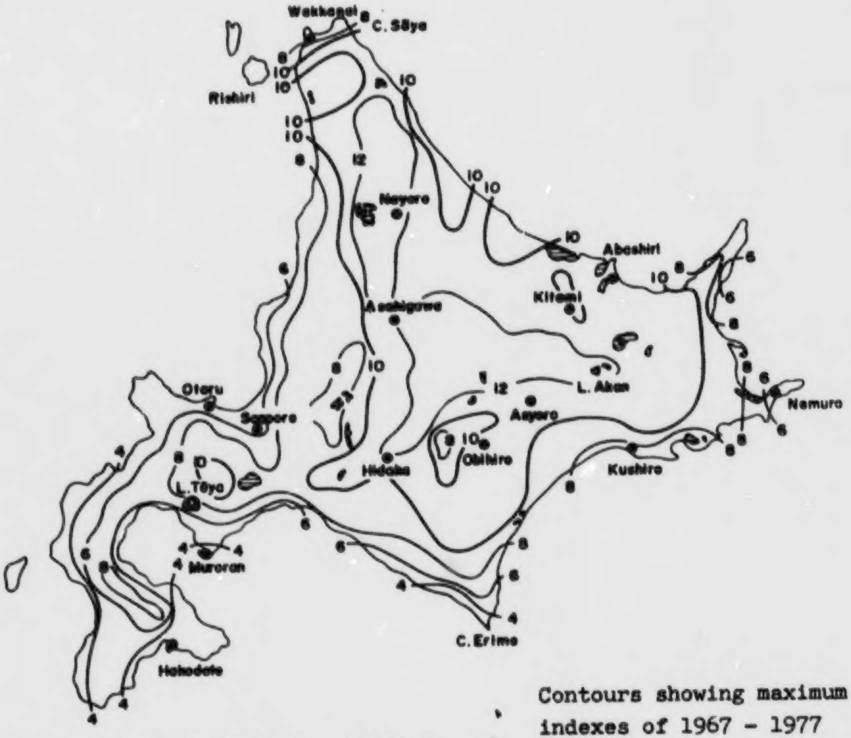

0.50
6.00
0.50
Shoulder
Surface Course t=4 cm
2%
2%
Shoulder
Asphalt Stabilized Base Course
t=6 cm
Subbase Course, Pit-run Gravel
t=47 cm
Sand
Sand
t=92 cm
Antifrost Layer, Sand t=35 cm
Insulating
Board
t=25 mm
Insulating
Board
t=25 mm
Frost Susceptible Subgrade Soil

Figure 11 Use of an Insulating Layer in Road Shoulder-portion

Figure 12 Bibi Test Road near the Chitose Airport in Hokkaido


Wakkanai
C. Sōya
Rishiri
Nayoro
Asahigawa
Abashiri
Kitami
L. Akan
Aayoro
Obihiro
Hidaka
Kushiro
Nemuro
C. Erimo
Otaru
Sapporo
L. Tōya
Muroran
Hakodate
12
10
8
6
4

Appendix

Freezing Index in Hokkaido (× 100°C·Days)

Contours showing maximum freezing indexes of 1967 - 1977

Session 3

The Val Gagne Pavement Insulation Experiment

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Ontario Ministry of Transportation and Communications

ABSTRACT

Since the first successful treatment of a frost heave problem in 1966 using Styrofoam* HI brand extruded polystyrene foam insulation, Ontario has used substantial quantities of insulation. However, knowledge of the performance of the insulation has been limited to sparsely documented observations of some differently insulated sites. In 1972, a joint experiment was launched by the Ministry and the Dow Chemical Inc. to construct, instrument, and observe the performance of pavement insulation at Val Gagne in Ontario.

Three thicknesses of insulation were used in the experiment, and four different taper designs. Winter temperature profiles, frost penetration depths, and frost heave measurements were observed during the winters of 1973 to 1977. The results of these observations were used to verify a two-dimensional finite element heat flow computer program intended for use in the design of insulation for controlling frost penetration. While the program accurately predicts ground temperature for uninsulated situations and thin insulated sections, changes in the program are needed to correct inaccuracies in prediction for thick insulations.

The results were also used in the development of a set of frost penetration prediction curves under various thicknesses of insulation and at locations with different degree-days of freezing temperatures. These curves may be used to select the appropriate insulation thickness for any acceptable depth of frost penetration.

Styrofoam insulation recovered from the test site after five years shows virtually no structural changes or changes in thermal properties.

* Trademark of the Dow Chemical, now called Dow Chemical Canada Inc.

THE VAL GAGNE PAVEMENT INSULATION EXPERIMENT

By

Tom Louie, Dow Chemical Canada Inc.

W.A. Phang and R.A. Chisholm

Ontario Ministry of Transportation and Communications

1/ INTRODUCTION

Differential frost heaving poses serious potential safety hazards (Figure 1) and has perennial after-effects on highways located in the northern areas of the United States and Canada. Prior to 1966, all treatments of frost heaves in Ontario involved removal of the frost susceptible soil to great depths and/or ditching and drainage to remove entrapped water and to lower the water table. These treatments were not always successful.

Research into the use of Styrofoam for highway insulation began in the United States in the early 1960s, and by 1965, several full-scale experimental projects had been constructed [1, 2, 3, 4]. These early research sites were instrumented with thermocouples and frost depth indicators to measure the insulating properties of various thicknesses of Styrofoam insulating. The first major research sites in Canada were constructed near Winnipeg on the Trans-Canada Highway in 1962, and near Sudbury in 1966 [5]. The findings of these early studies have resulted in the present widespread use of Styrofoam in areas that have sufficient frost penetration to make differential frost heaving a significant problem.

Since the 1970's, Ontario has been using increasing amounts of Styrofoam HI brand extruded polystyrene foam insulation for frost heave treatments in the northern part of the province. The reasons for this are two-fold.

(a) Experience has shown that correctly designed Styrofoam always works to prevent recurrence of the frost heave, whereas, other more conventional treatments may or may not be successful.

(b) Cost comparisons of the various alternatives tend to show Styrofoam as being the most economical treatment.

In Ontario, highway insulation has been adopted by the Ministry as an economical treatment for differential frost heaving. By 1972 the province was using approximately 3500 m^3 of Styrofoam per year [6] for treating frost heaves.

The thicknesses of Styrofoam used in early installations depended on the number of degree-days of frost at the heave location. The rule of thumb used in the design of these treatments was 25 mm of Styrofoam for each 555 degree-days Celsius of frost to prevent frost penetration. Through a trial and observation approach, some of the variables such as minimum depth of granular over Styrofoam and desired taper lengths were determined and incorporated in a set of design standards based on the best performing installations. Studies into the use of Styrofoam for highway insulation have been continued in recent years by regional staff, essentially in the form of close observation of the performance of numerous installations.

In 1971, the Ministry and Dow Chemical Canada Inc. jointly constructed a full-scale insulation research installation principally to investigate the most effective utilization methods for the insulation. This experiment was constructed during the summer of 1972 at Val Gagne, located 65 km south of Cochrane. The basic experiment involved the use of three thicknesses of Styrofoam HI-35 covered with 460 mm of granular and 75 mm of asphaltic concrete. Full instrumentation was installed to measure ambient and ground temperatures, pavement elevations (frost heave) and frost penetrations. This report details the construction, instrumentation and results of this five-year study (1972 to 1977).

2/ GOALS AND OBJECTIVES

The objective behind the setting up of the Val Gagne test site was to attempt to find solutions to problems in the current design standards.

The overall goal was to minimize specific problems presently encountered with insulation, through the evaluation of data from an experimental installation.

The long term goal was to best utilize insulation materials in highway construction from both an economical and technical viewpoint.
The following objectives formed the basis of the site's design:

2.1/ Pavement Heaving

In certain insulated sections completed in recent years, a differential frost heave had occurred at the transition from insulated to non-insulated sections. Several theories have been put forward to explain this problem but none are based on scientific data. From heave measurements taken in the three Styrofoam thicknesses used, as well as through the transition zones, it may be possible to minimize or eliminate the differential heave now associated with insulated sections.

2.2/ Depth of Frost Penetration

When Styrofoam was first used for insulation of highways, it was thought that to be effective the thickness should be sufficient as to stop all frost penetration below the Styrofoam layer. One of the prime objectives was to measure the depth of frost penetration in an area of 1850 degree-days Celsius of frost or greater under varying thicknesses of insulation.

The information obtained will permit designs with controlled frost penetration below the Styrofoam layer.

2.3/ Long Term Effects of Subgrade Insulation

The effect of Styrofoam insulation on the long term thermal regime below the layer is not clearly understood. The heat flow from the subgrade upwards is prevented during the winter months; however, the insulation also restricts heat flow into the ground during the summertime. It is possible that the seasonal fluctuation of temperature will be restricted below the STYROFOAM layer and the mean annual ground temperature may change due to the altered heat flow characteristics. The Research Site will hopefully shed some light on this subject.

2.4/ Pavement Icing

The occurence of pavement differential icing over insulated sections in the fall and occasionally in the spring of the year when surface moisture is available presents a potential serious hazard to the motorist. The altered heat flow properties in these areas keeps the moisture frozen over the treated pavement. It was intended at this site to closely monitor the three test sections, but due to the site location only haphazard icing observations were possible; icing is not dealt with further in this report.

2.5/ Frost Depth Indicator Reliability

Fifteen modified Swedish frost depth indicators [7] were installed in various locations throughout the site. Several of the indicators were placed within 0.6 m of the thermocouple sets. The comparison between the thermocouple readings and the frost depth indicator measurement will provide a check on the reliability of the indicators.

2.6/ Layer Width

The experiment will contribute to the knowledge of thermal conditions at the edge of insulation treatments and provide guidance in considering the possibility of reducing design widths.

2.7/ Computer Prediction Model

Dow engineers have recently developed a two-dimensional finite element computer model for prediction of heat flow characteristics in pavement design. This site will provide verification of the prediction accuracy of the program under the extreme environmental conditions which are experienced in Northern Ontario.

2.8/ Durability of Styrofoam

The durability of Styrofoam used for pavement insulation will be studied from samples taken after five years of use.

3/ LAYOUT OF THE EXPERIMENT

The test site is located on Highway 11 in the Cochrane District (Figure 2) approximately 3.2 km north of the junction of Highway 101. The site was selected for several reasons.

(a) The pavement at this location did not experience an abrupt or hazardous heave, but rather a general distortion during the winter months.

(b) The Val Gagne area regularly experiences severe winter weather with freezing indices of 1833 degree-days Celsius or greater. Frost indicator measurements have shown that penetration to a depth of 2.4 m to 3.0 m is common [7].

(c) The roadway was constructed as a 1.2 m to 1.5 m cut through a heavy, clay soil. This cut is uniform in depth throughout the site as the original ground profile has the same slope as the pavement.

(d) The site is 65 km south of the Cochrane District office which allows relatively easy access by staff assigned to take regular weekly measurements during the fall, winter and spring.

(e) The experiment was designed to evaluate three different thicknesses of Styrofoam: 25 mm, 50 mm and 75 mm. Each section is 29.2 m in length plus transitions, and is separated from each other by non-insulated control sections. Figure 3 shows the site layout.

4/ SOILS

Soil samples were obtained for various depths throughout the test site [8]. The field work consisted of ten sample boreholes. Four boreholes were taken 1.7 m east of the centreline. All boreholes were taken using a hollow-stemmed auger.

The laboratory tests on the soil samples have shown that the stratigraphy at the test site consists of a 100 mm layer of asphalt pavement on approximately 900 mm of granular underlain by a wet-to-saturated varved clay soil.

The subgrade soils at the site are mainly fat and gumbo clays. This soil is classified as CH soil according to the Unified Soil Classification System, or an A-7-6 type soil as specified by the AASHTO Soil System.

This soil has a very high plasticity index and a liquid limit greater than 50%. The water content of the silty clay soil ranged from 22% to 67%. The water table was 1.2 m below the pavement surface in three boreholes.

Under the Unified Soil Classification System, CH (silty clay) soil is considered to be a frost-susceptible soil, since more than half of the fine grained aggregate contained in the clay passes the #200 sieve.

Results of the tests taken on the subgrade soil are given in Table 1.

5/ DESIGN AND CONSTRUCTION

Design of the test sections was in accordance with the Ministry's standards SD-4-86, 87 and 91 [9], except for the taper lengths on the southern end of both the 50 mm and 75 mm sections. The width of the STYROFOAM layer was extended 1.8 m beyond the pavement edge (Figure 4).

The 50 mm thick section had the normal 4.9 m transition on the northern or uphill end and a longer 7.3 m length of 25 mm insulation on the other end (Figure 3). A longer, 12.2 m transition consisting of 4.9 m of 50 mm and 7.3 m of 25 mm foam was placed at the southern end of the 75 mm section. The normal 9.8 m transition was used at the other end.

The 2.40 m by 0.60 m Styrofoam boards of the required thickness were laid directly on the old pavement surface after a light tack coat of emulsion had been applied to the surface by hand spray. Shovels filled with granular base material were placed on the Styrofoam boards to prevent movement during the subsequent operation (Figure 5).

The granular base material was end dumped at the beginning of the Styrofoam layer and spread in a thick layer by a small bulldozer over the length of the section. With this construction method there is minimal displacement of the Styrofoam sheets. A minimum layer thickness of 310 mm of granular material is placed on top of the Styrofoam to prevent its damage by subsequent compaction and other operations by construction equipment. The completed sections were brought up to profile grade with a total lift of 460 mm of granular cover and then paved with 75 mm of HL-4 asphaltic concrete. A more detailed description of the construction is contained in Reference 10.

6/ INSTRUMENTATION

Full site instrumentation was installed to measure ambient ground temperatures, frost penetration and pavement heaving. Ground temperatures were taken by the use of 21 copper constantan thermocouples installed in each insulated section and control section. The thermocouples were arranged in three vertical strings with one set placed at the centreline, one at the edge of pavement and one at the edge of the Styrofoam layer. The strings have thermocouples at the following depths: top and bottom of the pavement, above the Styrofoam, below the Styrofoam, 0.3 m below the Styrofoam, and three at 0.61 m intervals below that. A total of 84 thermocouples were installed in this way (Figures 4 and 6).

In addition to this, three thermocouples were placed in the 25-mm transition of the 75 mm section; one thermocouple was located under a "finger," one between the "fingers," and one off the end of the "finger." (In transitions, every other sheet is extended beyond the body of the insulation, and is called a "finger.")

Originally the thermocouple wires were run into wooden boxes mounted on posts by the edge of the shoulder, and rotary switches were used to connect the individual thermocouples to the readout potentiometer for measurements. This method was used during the 1972/73 winter season. However, thermocouple readings taken at different times during this period were found to be inconsistent and erratic. It was felt that the problem was condensation forming inside the switch boxes when they were moved from their unheated wooden boxes into a warm vehicle interior for the measurements to be taken. To solve this problem, a heated utility shed (1.8 m by 2.4 m) was erected on site and all thermocouples led into this building. No problems were encountered after this change, and the majority of thermocouples continued to operate normally.

During the summer of 1973, a Stephenson screen and thermohydrograph were installed on site to make possible continuous recording of both ambient air temperature and relative humidity.

Fifteen frost depth indicators of the modified Swedish type were located throughout the site (Figures 3 and 7). They were installed in the middle of each section, with some being placed in the tapers in order to measure frost penetration through this critical area. These frost depth indicators have been used by the Ministry for several years and are an inexpensive and efficient way to measure frost depth.

In order to facilitate accurate monitoring of the heaving movements occurring throughout the test site, approximately 700 nails were placed in the pavement (Figures 8 and 9). These nails were set in epoxy in pre-drilled holes in the pavement surface. Two benchmarks were also placed in the pavement at the site. These benchmarks are founded well below the maximum frost depth and consist of steel rods in outer telescoping casings, to ensure that the elevation will remain constant throughout the years. A geodetic benchmark located near the site will provide an accurate datum for all measured elevations.

The nails were spaced at 1.2 m longitudinal intervals throughout the site. Through all transition areas, the nails were spaced longitudinally 0.61 m apart extending 6.1 m in each direction beyond the transitions. Three rows were placed in this manner at the transitions -- along centreline, and in the centre of each lane.

6.1/ Measurement Frequency

A five-year period of observation of the test site has been completed. During this time, the thermocouples were normally read weekly during the frost season and at monthly intervals over the summer months. Frost depth indicators were normally read weekly when any ground frost was present. Four sets of elevations on the nails were taken yearly: every fall before any frost enters the ground, at mid-winter, during the spring thaw, and mid-summer.

7/ TEMPERATURE AND PRECIPITATION

Only 290 km north of the Val Gagne test site lie the shores of James Bay and arctic tidewater. With an average annual air temperature of only 1°C [11] and winter temperatures often dropping to below -40°C, severe winters are not an uncommon experience. Ambient air temperatures and precipitation for the site were obtained from the Environment Canada Climatological Station at Iroquois Falls, located approximately 24 km to the northeast. Additional backup data was available from the station in Cochrane and on-site data was provided by the thermohydrograph installed in 1973. An indication of the type of weather found at this site is shown in Figure 10 in which the average monthly temperatures and ranges recorded at Iroquois Falls station are plotted.

Freezing indices calculated using the mean daily air temperatures as well as the coldest recorded air temperatures in each of the five years are tabulated in Table 2.

The weather during the five-year test period ranged from a very severe winter experienced during 1973/74 to the relatively mild winter of 1974/75. This range provided a good test for the materials and their performance under varying climatic conditions.

Table 3 lists the monthly precipitation recorded at Iroquois Falls during the test period.

8/ GROUND TEMPERATURES

Temperatures in each section were taken at various depths (Figure 4) by the use of thermocouples. During the winter months temperatures above the insulation are usually considerably lower than those below. This temperature difference increases with greater insulation thickness. A typical example of temperatures taken above and below the 50 mm insulation layer is illustrated in Figure 11.

Complete results of temperature measurements are reported in References 12, 13 and 14.

9/ FROST PENETRATION TIME PROFILE

Frost penetration can be determined through interpolation of the thermocouple readings or by direct reading from the frost indicators. Figure 12 shows typical time-depth patterns of frost penetrations, as determined by frost indicators at the pavement centreline in all sections. The insulating effect of the Styrofoam layer is clearly illustrated, with increasing thickness of Styrofoam leading to reduced frost penetration. Frost penetration, as recorded by the indicators for all sections over the five years, is summarized in Table 4. Greater depths of frost were recorded in each succeeding year except for the very mild winter of 1974/75. The reason for this continuously increasing penetration is not known, although it may be partially due to the lowering of subsoil temperatures under the Styrofoam layer, or by changes in subsoil moisture conditions. While there was a slight increase in moisture content of the Styrofoam (loss in insulating value), this would not account for the continuous increase.

It is important to note that some thermocouple data was too erratic for establishing the 0°C isotherm. An example of this is shown in Figure 13, where the frost penetration below the 25 mm layer of Styrofoam fluctuated from about 1.02 m on March 15 to 2.46 m on March 28. This rapid change in only a two-week period is highly unlikely and this suggests that errors were occurring either in instrument calibration or readout. The frost depth indicator, on the other hand, with its simplicity of design and use, shows a more consistent and logical progression of the freezing front.

9.1/ Transverse Cross Section of Frost Penetration

The depths of frost penetration as measured transversely across each section are illustrated in Figure 14. In both the uninsulated and 25 mm sections, the depths of the 0°C isotherm as recorded by the thermocouples were much lower below the pavement surface at the centreline than at the edges of the embankment. This heat gain can be attributed to the insulating effect of the snow accumulated on the shoulders and ditches of the highway embankment. In the 50 mm and 75 mm sections, the 0°C isotherms recorded below the insulating layers were roughly parallel to the roadway surface.

9.2/ Maximum Frost Penetration

The maximum frost penetration recorded for each of the five years in the test sections and transitions along the length of the site is illustrated in Figure 15. The freezing front reached into the frost-susceptible subgrade soil under the 25 mm section every year. Only during the 1976-77 winter did the frost level under the 50 mm section come within a few inches of the subgrade. The frost level in the 75 mm insulated section remained well within the gravel base layer in all years.

10/ FROST HEAVING

Soil conditions at this site did not lead to unusual or severe differential frost heaving. In general, during the more severe winters, the uninsulated sections of roadway heaved approximately 24 mm to 40 mm. This compares with heaves ranging from 0.0 mm to 6 mm in the 75 mm sections, and 8 mm to 24 mm in the 25 mm section. Table 5 lists the average heave for each section during the five-year test period. The slight bumps that did occur over the transition areas were never great enough to adversely affect the riding comfort over the test sections. Figure 16 illustrates the results at the centreline of surface measurements on both the insulated and uninsulated sections for the year 1973/74, 1974/75 and 1975/76. Further information such as heave profiles is shown for centreline and midlanes in References 12, 13, and 14.

10.1/ Transverse Heaves

Heave measurements taken transversely across the pavement clearly show the difference in heaving between the insulated and uninsulated sections. The pavement edge tends to heave more than the centre of the pavement, possibly due to water penetrating under the pavement edge. Typical results of the transverse heave measurements are shown in

Figure 17. The heave is negligible over the 50 mm and 75 mm layers of Styrofoam.

10.2/ Transition Heaves

The transition tapers on the southern end of the 50 mm and 75 mm sections were lengthened to try to provide a more gradual transition from the insulated sections (which move relatively less) to the non-insulated portion of the highway (which moves relatively large amounts). Differential heaving was not a problem at this site as gradual changes rather than abrupt heaves occurred at all transitions.

It appears from the slope of the transition that the 50 mm section with the 7.3 m transition is better than the 4.9 m transition used on the north end. On the 75 mm section, no improvement was observed with the 12.2 m taper used on the south end over the standard 9.8 m taper on the north side. Figure 18 illustrates the 1973/74 heaves through the transition areas.

11/ LONG-TERM SUBGRADE TEMPERATURE REGIME

One objective of this study was to ascertain if long-term temperature changes were occurring in the subgrade below the Styrofoam sections.

Table 6 compares the subgrade temperatures of each Styrofoam section in relation to the uninsulated control section.

Measurements taken at 0.86 m below the surface show that the soil temperatures do not fluctuate as much from season to season as in the control sections. Data from the deepest thermocouples at 2.69 m exhibit a similar trend although the temperature difference between the insulated and uninsulated sections increases as Styrofoam thickness increases. The effects on the pavement structure of this temperature variance are not completely known; moisture migration effects, however, may be one area which should be explored more fully.

12/ SUBGRADE MOISTURE CONDITIONS

A detailed foundation investigation report [8] on the subsoil conditions found at the test site was completed by the Ministry's Soils Mechanics Section before construction began in 1972. In September 1976, the Geotechnical Office of Northern Region carried out another series of sampling and tests to determine if subgrade moisture conditions had changed as a result of the insulation being a part of the pavement structure. However, the accuracy of these latter results appears questionable because of the sampling method; therefore, no conclusions have been drawn for moisture movement.

13/ PAVEMENT CRACKING

In northern Ontario, thermally-induced transverse cracks are prevalent. A crack survey taken on the 75 mm pavement over the test sections exhibited this same pattern (Figure 19). Some centreline construction-joint cracks were also present. There was no significant difference between sections, as all insulated and non-insulated had major cracks both in the body of the section and in the transition areas. The uniform crack patterns do not reveal any differences in performance due to different thicknesses of insulation.

14/ DURABILITY OF STYROFOAM INSULATION

Three different thicknesses of Styrofoam samples (25 mm, 50 mm and 75 mm), approximately 0.45 m square, were recovered near the wheel path of the driving lanes of the highway after being buried under the roadway for five years. The recovery procedure involved removing 75 mm of asphaltic concrete pavement and 460 mm of granular base course. Two samples (25 mm and 75 mm) were removed from the northbound driving lane and the remaining one sample (50 mm) was recovered from the southbound lane.

All recovered samples were immediately sealed in polyethylene bags for transporting to the laboratory. Before laboratory testing, dirt on the surface of the samples was carefully removed by washing.

Table 7 summarizes the laboratory results of tests on the three Styrofoam samples removed from the pavement.

Only in the 25 mm thick sample was there an observed loss in measured thermal conductivity in the wet state. As for the 50 mm and 75 mm thick samples, both showed negligible moisture pick-up and, hence, no significant change in thermal conductivity after all these years. Furthermore, all three test samples had exhibited no decrease in thickness and no reduction in minimum vertical compressive strength of 240 kPa.

15/ COMPUTER ANALYSIS

The ability to accurately predict the depth of frost penetration beneath highways with or without insulation can be a valuable tool for the designer. A part of this project consisted of testing the accuracy of a computer prediction program developed by Dr. F.S. Wang of Dow Chemical Canada Inc. [15].

The program not only provides detailed predictions of frost depths and ground temperatures, but can be used for designing for the worst possible conditions or for determining the optimum designs for new pavement structures using experimental materials.

The model used is described as a two-dimensional finite element program for transient heat flow conduction. The model is capable of handling a multi-layered soil profile, with layers of varying thicknesses and thermal properties.

Computer simulations were run for all of the Val Gagne sections based on 1975/76 actual weather data. Complete details regarding these simulation runs can be found in Reference 15.

15.1/ Input Data

The 1975-1976 climatological data required for the simulation was obtained from the Climatological Station at Iroquois Falls. Soils information available from the Soils Report [8] was sufficient to characterize the thermal properties of the embankment and foundation soils. Thermal properties of the soils and insulation layers used in the simulation are listed in Table 8.

15.2/ Temperature Distribution Results

Figure 20 is an example of the comparison between the observed and computed temperatures directly above and below the 75 mm insulating layer. As clearly shown, the prediction model correlates well with the thermocouple results when the actual temperature pattern is used. Variations in the results can be caused by many factors, such as the effect of snow cover, variation of soil types and time-temperature distribution.

15.3/ Frost Penetration

The comparison between the predicted and the measured maximum penetration of the 0°C isotherm is shown in Figure 21 for the 25 mm and 75 mm sections, and the results for all sections are given in Table 9. An examination of the data shows that the computer prediction agrees well with the measured penetration in both the non-insulated control and the 25 mm sections. However, the program predicts almost identical frost penetration for both the 25 mm and 50 mm sections. This is not in agreement with measurements from the frost depth indicators which show not only less penetration but a substantial difference in penetration between the 25 mm and 50 mm sections as well.

An example of the predicted and measured frost penetration in the transverse direction is shown in Figure 22. It can be seen that the simulated pattern of frost penetration corresponds quite closely with the measured penetration.

In summary, the computer program now available closely predicts the frost penetration beneath the 25 mm Styrofoam and uninsulated sections. The program over-predicts the frost penetration in the thicker 50 mm to 75 mm Styrofoam sections.

16/ DESIGN OF STYROFOAM TREATMENTS

On the basis of the measurement of frost penetration at Val Gagne and other sites with Styrofoam insulation, a diagram of frost penetration versus thickness of Styrofoam insulation for different temperature environments was created. (Refer to the chart in Figure 23.) The frost penetration with no insulation was calculated from the prediction formula:

$$P = -0.328 + 0.0578 \quad F$$

and represents a five-year average value [7]. This chart allows the designer to determine the insulation thickness needed to "control" the penetration of frost to any desired depth, such as needed to control frost heaving.

For example, in an environment of 2000 degree-days Celsius, if frost may penetrate 1.5 m without excessive heave, 40 mm is the standard thickness of insulation needed for the treatment.

17/ CONCLUDING REMARKS

(a) Experience has shown that frost depth probes are more reliable indicators of the location of the frost front than instantaneous thermocouple readings. Thermocouple readings taken on a continuous automatic basis will give reliable frost penetration results.

(b) Longer lengths of transition tapers do result in a reduction in the abruptness of the bump at the end of the insulation caused by differential heave. However, in this experiment, the differential heave was not very large and no recommendation is made to increase standard taper length.

(c) Frost measurements made transversely indicate that reducing the standard width of the insulation beyond the pavement edge from 1.8 m to 1.2 m would not substantially alter the frost penetration profile.

(d) At this site, a depth of cover of 460 mm of gravel base adequately protects the insulation from the effects of construction equipment and traffic, so that there was no detectable change in the final insulation thickness. As well, over the five years of tests, only the 25 mm-thick insulation showed any detectable change in thermal insulating value.

(e) The insulation not only retards the escape of ground heat to the air during winter, it retards the replenishment of ground heat storage during the summer months. However, the observations at this site are not adequate to show effects of reduced heat storage on frost penetration.

(f) The computer simulation program accurately predicts the frost penetration of uninsulated pavements and thin (25 mm) insulated sections. The computer predicts deeper frost penetration than actual for the thicker (50 mm, 75 mm) insulated sections. The measurements at the site can be used to check future modifications to the computer program made to more accurately predict penetration for thicker insulation.

18/ RECOMMENDATIONS

(a) The chart for frost penetration under different thicknesses of Styrofoam insulation may be used to select insulation for controlling frost heave.

(b) Further refinements which are needed to the computer simulation program for the thicker lifts of insulations should be examined.

(c) The standard width of insulation may be reduced by 0.60 m on either side of the cross sections.

(d) Futher tests should be carried out on recovered samples of the 25-mm thick insulation.

19/ DEFINITIONS

Degree-Days

The degree-days for one day equals the difference between the average daily air temperature and 0°C. The degree-days are minus when the average daily temperature is below 0°C (freezing degree-days), and plus when above 0°C (thawing degree-days).

Freezing Index

The number of degree-days (above and below 0°C) between the highest point in the autumn and the lowest point the next spring on the cumulative degree-day time curve for one freezing season.

Mean Freezing Index

The freezing index determined on the basis of mean temperatures. The period of record over which temperatures are averaged is usually a minimum of ten years (preferably 30) and should be the latest available.

Design Freezing Index

The average air freezing index of the three coldest winters in the latest 30 years of record. If 30 years of record are not available, the air freezing index for the coldest winter in the latest ten-year period may be used.

Average Daily Temperature

The average of the maximum and minimum temperatures for one day or the average of several temperature readings taken at equal time intervals during one day.

Frost Heave

The raising of a surface due to formation of ice in the underlying soil.

Plasticity Index

The difference between the liquid limit and the plastic limit. PI is an indication of the change that can take place in the moisture content of the soil before the soil goes from a semi-solid to a liquid condition.

Liquid Limit

The water content at which a soil passes from the plastic to the liquid state.

Plastic Limit

The moisture content at which a soil changes from a semi-solid to a plastic state.

Bench Mark

A mark on a permanent object indicating elevation and serving as a reference for future elevations. The bench marks are not affected by frost action.

Thermohydrograph

A mechanical device used in taking continuous measurements of temperature and humidity.

Thermocouple

A thermoelectric couple used to measure temperature differences.

Thermal Conductivity ("K" Factor)

Time rate of transfer of heat by conduction through unit thickness, across unit area for unit difference of temperature.

0° Isotherm

The depth in the pavement structure at which the soils change from a frozen to an unfrozen state. This would represent the maximum frost penetration at the time the readings were taken.

Volumetric Heat Capacity

The quantity of heat required to change the temperature of a unit volume by one degree Celsius.

Latent Heat of Fusion

The quantity of heat required to melt the ice (or freeze the water) in a unit volume of soil without a change in temperature.

Frost-Susceptible Soil

Soil in which significant detrimental ice segregation will occur when the requisite moisture and freezing conditions are present.

Nonfrost-Susceptible Soil

Noncohesive materials such as crushed rock, gravel, sand, slag, and cinders in which significant detrimental ice segregation does not occur under normal freezing conditions.

Stevenson Screen

A small box used to protect meteorological instruments in the field.

21/ REFERENCES

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[7] R.A. Chisholm and W.A. Phang, "Measurement and Prediction of Frost Penetration in Highways," Ministry of Transportation and Communications, Ontario, September 1978.

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[9] "Transportation Systems Design Standards Manual," Design Division, Ministry of Transportation and Communications, Ontario.

[10] R.B. Adamson, "Construction Report on the Styrofoam Research Site in Cochrane District," Materials and Testing, Northwestern Region, September 1972.

[11] D.W. Boyd, "Normal Freezing and Thawing Degree-Days for Canada 1931 - 1960," Environment Canada, C114-73, 1973.

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[13] T.M. Louie, "Val Gagne Test Site, Second and Third Year's Observations of an Insulated Highway Embankment," Construction Materials Section, Research and Development, Dow Chemical Canada, Inc.

[14] T.M. Louie, "Val Gagne Test Site, Fourth year Observations of an Insulated Highway Embankment," Construction Materials Section, Research and Development, Dow Chemical Canada, Inc., July 12, 1978.

[15] T.M. Louie, "Val Gagne Test Site, Comparing the Predicted and

"Theoretical Results and the Actual Measured Data Using a Two-Dimensional Thermal Computer Program," Dow Chemical Canada, Inc., November 6, 1978.

Table 1/ Physical Properties of the Subgrade Soils at the Time of Construction

BORE HOLE NUMBER	ELEVATION DEPTH BELOW PAVEMENT (metres)	MATERIAL NAME	DRY/ DENSITY t/m^3	WATER CONTENT (%)	LIQUID LIMIT (%)	PLASTIC LIMIT (%)	PLASTICITY INDEX (%)	CLASSIFICATION	
								A.A.S.H.O. SOIL	UNIFIED SOIL
1	1.5 – 2.4 2.4 – 4.8	SILTY CLAY SILTY CLAY	1.314 0.977	38 63	58 62	34 25	24 37	A – 7 – 5 A – 7 – 6	MH–OH CH
2	1.5 – 2.4 2.4 – 4.8	SILTY CLAY SILTY CLAY	1.668 1.073	22 52	51 59	26 29	25 30	A – 7 – 5 A – 7 – 6	CH CH
3	1.5 – 2.4 2.4 – 4.8	SILTY CLAY SILTY CLAY	1.666 0.881	16 66	34 66	26 28	8 38	A – 7 – 6 A – 7 – 6	ML–OL CH
4	1.5 – 2.4 2.4 – 4.0	SILTY CLAY SILTY CLAY	1.426 0.883	31 62	58 64	27 26	31 38	A – 7 – 6 A – 7 – 6	CH CH
5	1.5 – 2.4 2.4 – 4.8	SILTY CLAY SILTY CLAY	1.281 0.977	38 64	62 68	28 27	34 41	A – 7 – 5 A – 7 – 6	CH CH
6	1.5 – 2.4 2.4 – 4.8	SILTY CLAY SILTY CLAY	1.041 1.041	59 66	65 67	27 27	38 30	A – 7 – 6 A – 7 – 6	CH CH
7	1.5 – 2.4 2.4 – 4.8	SILTY CLAY SILTY CLAY	1.468 0.977	27 63	58 67	28 27	30 40	A – 7 – 6 A – 7 – 6	CH CH
8	1.5 – 2.4 2.4 – 4.8	SILTY CLAY SILTY CLAY	0.893 0.861	62 67	66 67	27 22	39 45	A – 7 – 6 A – 7 – 6	CH CH
9	1.5 – 2.4 2.4 – 4.8	SILTY CLAY SILTY CLAY	1.234 0.977	47 63	64 59	28 29	36 30	A – 7 – 6 A – 7 – 6	CH CH
10	1.5 – 2.4 2.4 – 4.8	SILTY CLAY SILTY CLAY	1.468 0.901	31 66	52 70	23 29	29 31	A – 7 – 5 A – 7 – 6	CH CH

Table 2/ Freezing Indices and Lowest Recorded Temperatures Taken from the Nearest Climatological Stations

	Climatological Station			
	Iroquois Falls		Cochrane	
Year	Degree-Days °C	Lowest Temp °C	Degree-Days °C	Lowest Temp °C
1972 – 1973	1810	-41	1806	-44
1973 – 1974	2097	-40	2209	-45
1974 – 1975	1659	-40	1783	-42
1975 – 1976	2038	-42	2124	-46
1976 – 1977	1849	-38	2080	-39
5-Year Mean	1891	-	2000	-
30-Year Mean	1862*	-	1936*	-
Design Freezing Index	2097	-	2209	-

*Reference (11)

Table 3/ Precipitation over the Five Year Test Period Taken at Iroquois Falls Climatological Station

YEAR	JAN		FEB.		MAR.		APR.		MAY		JUNE		JULY		AUG.		SEPT.		OCT.		NOV.		DEC.		TOTAL	
	R	S	R	S	R	S	R	S	R	S	R	S	R	S	R	S	R	S	R	S	R	S	R	S	R	S
1972		71.4		28.4	1	7.11	7	3.8	61	17.8	103		77		87		71		66	4.1	29	17.8		80.0	80.1	281.4
1973	10	38.4		38.9	77		40	2.0	72	5.8	66		70		99		117		84		20	14.0		46.5	634	133.6
1974		86.6		36.1	13	28.1	17	84.1	63	6.7	71		122		113		108		74		10	6.4		52.1	[illegible]	234.1
1975	19	52.8	9	18.8			41	7.6	70		9		70		60		77		53		34	67.9	16	40.6	480	175.4
1976		64.8		28.2	41	86.4	53	22.9	36	7.6	43		56		99		62		33	9.1		35.3		68.7	383	280.0

R – RAILFALL MILLIMETRES
S – SNOWFALL CENTIMETRES

Table 4/ Maximum Frost Penetration in Metres as Measured by the Frost Depth Indicators.

	Frost Penetration in Metres				
Section	1972-73	1973-74	1974-75	1975-76	1976-77
Control no insultation	2.36	2.49	2.39	2.59	2.62
25 mm insulation	1.60 (1.07)	1.73 (1.20)	1.60 (1.07)	1.90 (1.37)	2.03 (1.50)
50 mm insulation	1.02 (0.49)	1.14 (0.61)	1.12 (0.59)	1.22 (0.69)	1.42 (0.89)
75 mm insulation	0.58 (0.05)	0.94 (0.41)	0.68 (0.15)	1.02 (0.49)	1.09 (0.56)

Numbers in parenthesis are measurements taken below the STYROFOAM layer.

Table 5/ Average Heave Measured at the Centreline (mm)

Section	1972-73	1973-74	1974-75	1975-76	1976-77
Control no insultation	30	37	0	34	18
25 mm insulation	9	24	3	9	2
50 mm insulation	0	6	3	3	0
75 mm insulation	0	6	0	3	0

Table 6/ Changes in Subgrade Temperature as Compared to the Uninsulated (Control) Section Temperatures (°C)

Thermocouple Location	THERMOCOUPLE READING 0.26 m below Surface				*THERMOCOUPLE READING 2.85 m below Surface			
		Temp. Difference from Control Section				Temp. Difference from Control Section		
Date	Control Actual Temp °C	25 mm Section	50 mm Section	75 mm Section	Control Actual Temp °C	25 mm Section	50 mm Section	75 mm Section
August 1973	22	-4	-7	-8	16	-4	-6	-6
February 1974	-11	+5	+9	+11	-2	+3	+3	+4
August 1974	22	-5	-8	-10	15	-3	-5	-6
February 1975	-13	+6	+10	+11	-3	+4	+5	+5
May 1975	14	-1	-1	-1	14	-3	-2	-2
February 1976	-11	+4	+8	+10	-3	+3	+4	+5

Table 7/ Laboratory Test Results of STYROFOAM® Samples

Sample Identification	'K' Factor (wet) ASTM C-518 $\frac{W}{m^2 - 0^\circ C}$	'K' Factor (dry) ASTM C-518 $\frac{W}{m^2 - ^\circ C}$	Water Content % by Volume	Dry Density FP-3 kg/M³	Vertical Compressive Strength @ 5% Defl.		Vertical Compressive Modulus kPa
					kPa	@ 10% Defl. Kpa	
25 mm STYROFOAM HI-35	1.329	1.203	3.95	37.49	291.7*	337.9*	8964*
50 mm STYROFOAM HI-35	1.260	1.203	1.29	40.05	519.2	677.8	12742
75 mm STYROFOAM HI-35	1.243	1.197	0.93	33.00	344.1	344.1	13204

* Irregularity in surfaces were removed.

Table 8/ Thermal Properties of Embankment Layers

Material Layers		Dry Density kg/m³	Water Content %	Thermal Conductivity $\frac{W^{\circ}}{m^3\ ^{\circ}C}$		Volumetric Heat $\frac{KJ}{m^3\ ^{\circ}C}$		Latent Heat of Fusion KJ/m³
				Frozen	Thawed	Frozen	Thawed	
Asphaltic Concrete		2371	0	58.60	58.60	3078.3	3078.3	16643
Gravel Base		2211	6.5	86.17	86.17	2608.9	2917.4	48138
STYROFOAM HI-35		36.8	0	1.1356	1.1356	48.29	48.29	0
Subgrade No. 1	Control Section	1698	22	87.21	66.41	2629.0	4124.6	126116
(Silty Clay)	25 mm	1394	38	96.07	51.78	2682.6	4319.1	177380
1.5 m to 2.4	50 mm 75 mm	1666 1458	15 31	59.96 88.56	53.15 52.46	2664.0 2608.9	3654.5 4091.0	83684 161346
Subgrade No. 2	Control Section	1089	51	87.90	36.11	2902.2	3755.7	184693
(Silty Clay)	25 mm	1009	61	96.07	34.07	2682.0	4286.5	166968
2.4 m to 7.6 m	50 mm 75 mm	981 977	67 62	98.80 92.66	32.71 32.71	2629.0 2608.9	4600.1 4426.4	216320 206200

Table 9/ Computer Prediction of Maximum Frost Penetration as Compared with Site Measurements

Section	Predicted 0°C Isotherm depth (m)		
	Thermocouples	Frost Indicators	Computer (daily air)
Control no insulation	3.10	2.59	2.59
25 mm insulation	2.71 (2.18)	1.90 (1.37)	1.93 (1.40)
50 mm insulation	2.08 (1.55)	1.22 (0.89)	1.90 (1.37)
75 mm insulation	1.65 (1.12)	1.02 (0.48)	1.47 (0.94)

Numbers in parenthesis represent measurements taken below the STYROFOAM level.

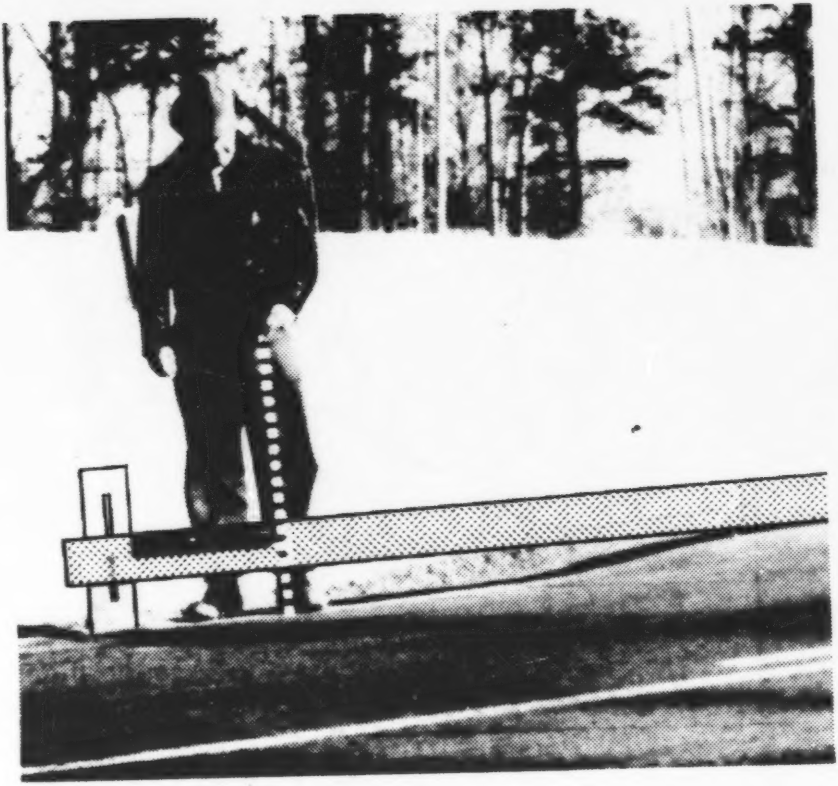


Figure 1/ Typical Frost Heave Considered for Treatment with STYROFOAM®

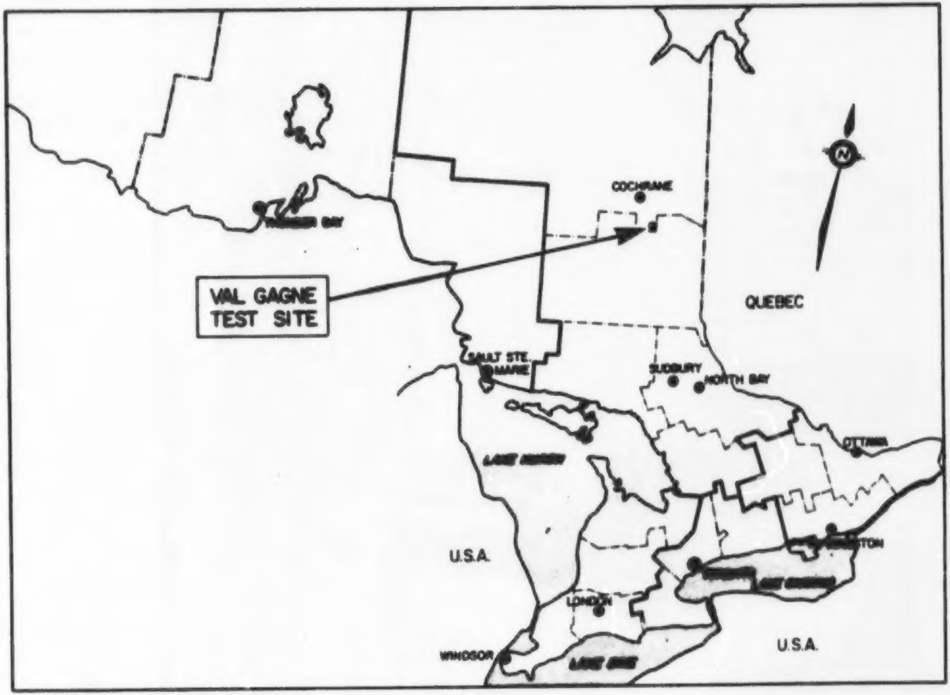

VAL GAGNE
TEST SITE
COCHRANE
QUEBEC
SUDBURY
NORTH BAY
OTTAWA
KINGSTON
SAULT STE. MARIE
LONDON
WINDSOR
U.S.A.
U.S.A.

Figure 2/ The Val Gagne Test Site

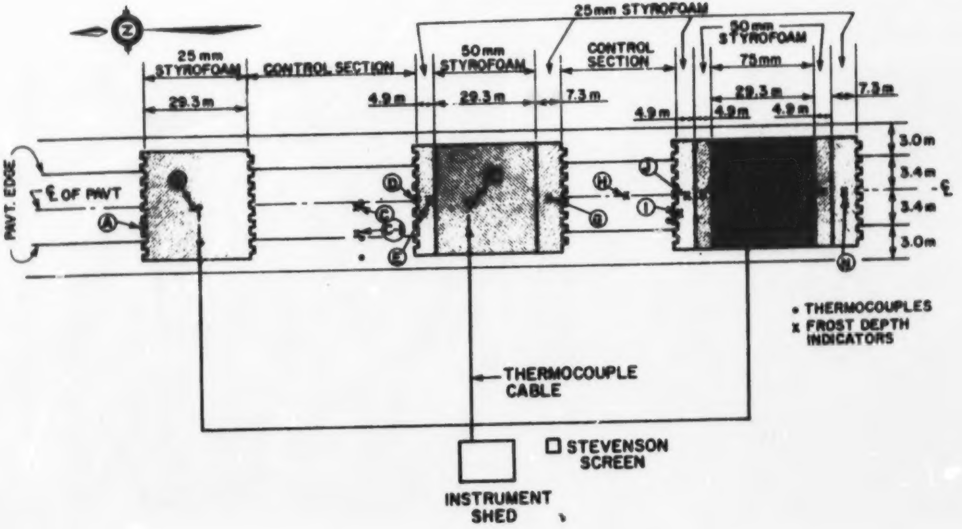
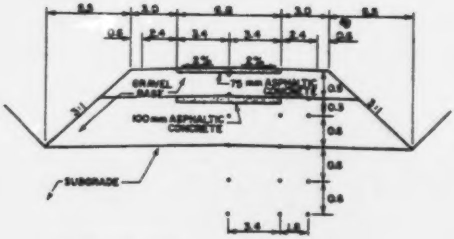
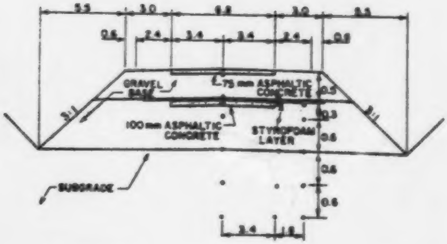

25mm STYROFOAM
25 mm STYROFOAM
CONTROL SECTION
50 mm STYROFOAM
CONTROL SECTION
50 mm STYROFOAM
75mm
29.3 m
4.9 m
29.3 m
7.3 m
4.9 m
29.3 m
4.9m
4.9 m
7.3 m
3.0 m
3.4 m
3.4 m
3.0 m
℄
PAVT. EDGE
℄ OF PAVT
A
C
D
E
G
H
I
J
K
• THERMOCOUPLES
x FROST DEPTH INDICATORS
THERMOCOUPLE CABLE
☐ STEVENSON SCREEN
INSTRUMENT SHED
N

Figure 3/ Val Gagne Test Site Layout


5.5
3.0
6.8
3.0
5.5
0.6
2.4
3.4
3.4
2.4
0.6
2%
2%
GRAVEL BASE
75 mm ASPHALTIC CONCRETE
100 mm ASPHALTIC CONCRETE
3:1
3:1
0.5
0.3
0.6
0.6
0.6
SUBGRADE
3.4
1.8

(a) NO INSULATION

• THERMOCOUPLES


5.5
3.0
6.8
3.0
5.5
0.6
2.4
3.4
3.4
2.4
0.6
GRAVEL BASE
75 mm ASPHALTIC CONCRETE
100 mm ASPHALTIC CONCRETE
STYROFOAM LAYER
3:1
3:1
0.5
0.3
0.6
0.6
0.6
SUBGRADE
3.4
1.8

(b) INSULATION

Note: All dimensions are in metres unless otherwise specified.

Figrue 4/ Typical Cross Sections, Uninsulated and STYROFOAM® Insulated Sections Showing Thermocouple Locations

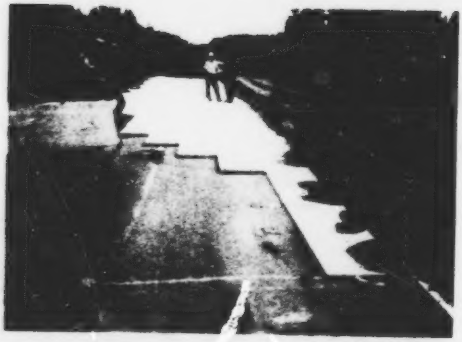


Figure 5/ Construction of the STYROFOAM® Insulated Sections

Figure 6/ Thermocouples Being Installed

Figure 7/ Frost Depth Indicators Being Installed

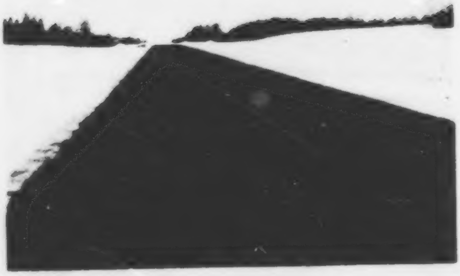


Figure 8/ Overall View of the Test Site Showing Levelling Nails and Instrument Shed

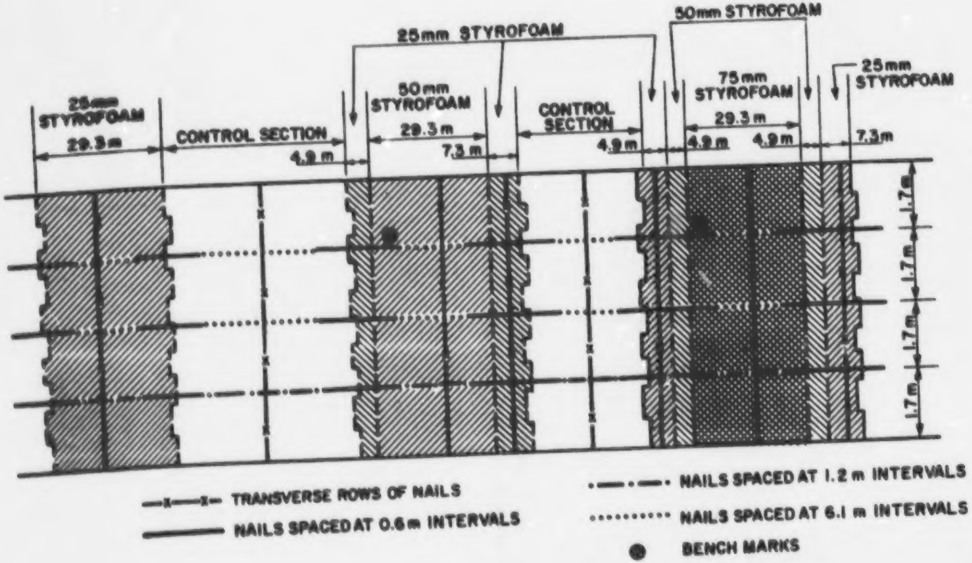

50mm STYROFOAM
25mm STYROFOAM
25mm STYROFOAM
29.3m
CONTROL SECTION
4.9 m
50mm STYROFOAM
29.3 m
7.3 m
CONTROL SECTION
4.9 m
75mm STYROFOAM
29.3 m
4.9 m
4.9 m
25mm STYROFOAM
7.3m
1.7m
1.7m
1.7m
1.7m
TRANSVERSE ROWS OF NAILS
NAILS SPACED AT 0.6m INTERVALS
NAILS SPACED AT 1.2 m INTERVALS
NAILS SPACED AT 6.1 m INTERVALS
BENCH MARKS

Figure 9/ Levelling Nails and Benchmark Layout

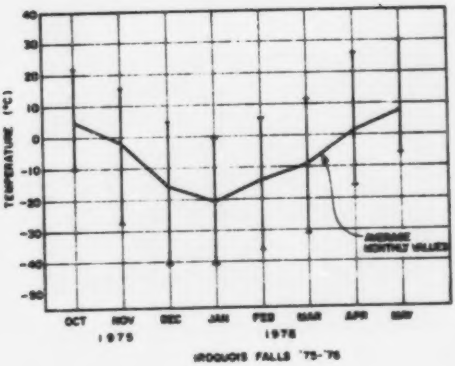


Figure 10/ Typical Monthly Maximum and Minimum Temperatures Recorded at Iroquois Falls 1975-1976

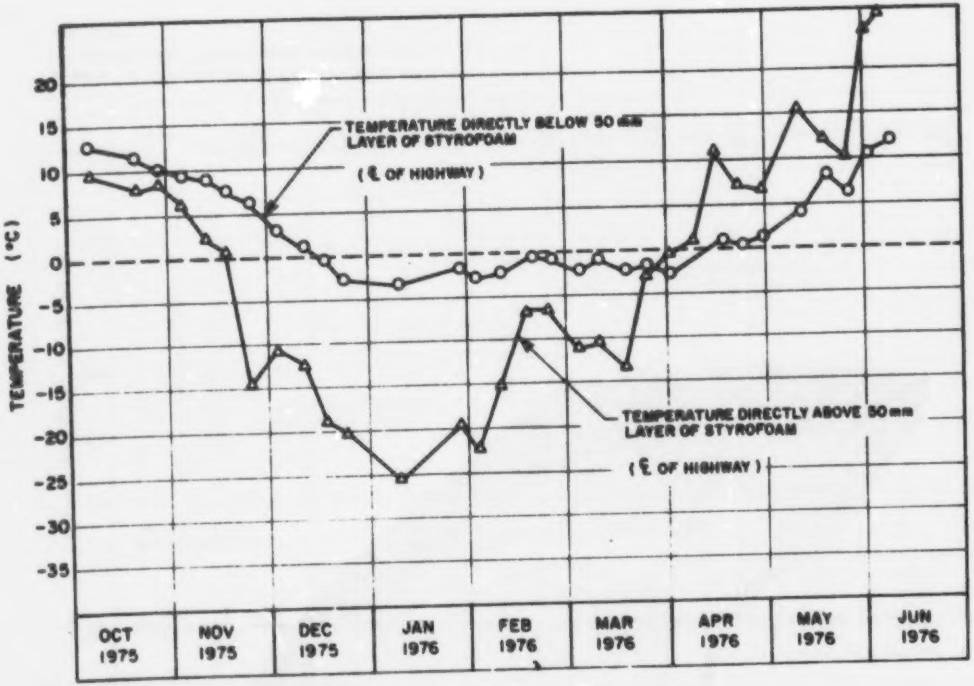


Figure 11/ Typical Temperatures Taken Directly Above and Below the 50 mm Layer of Insulation at the Centreline

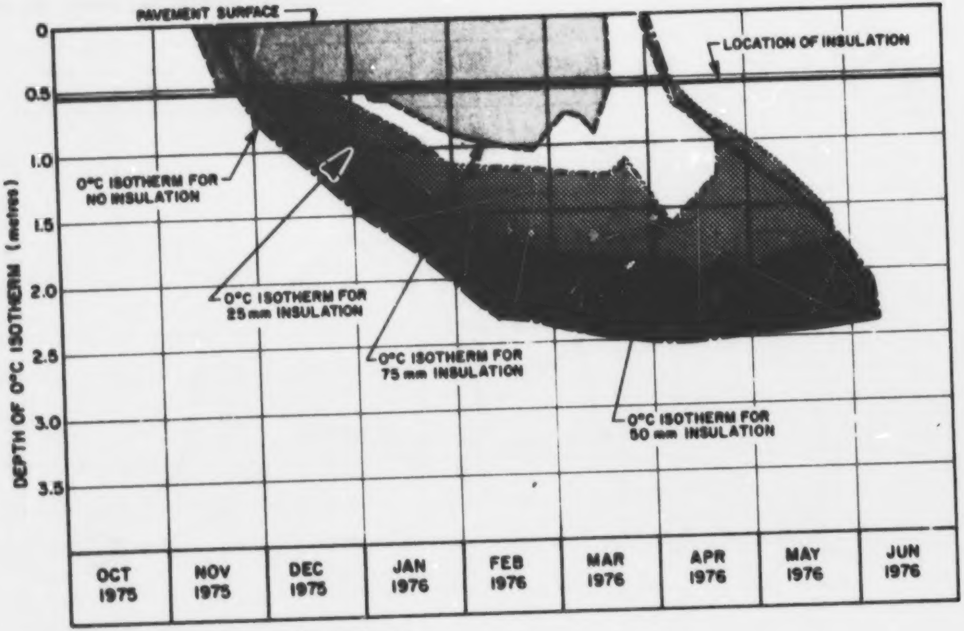


Figure 12/ Comparison of Frost Penetration as Recorded by the Frost Depth Indicators for all Sections

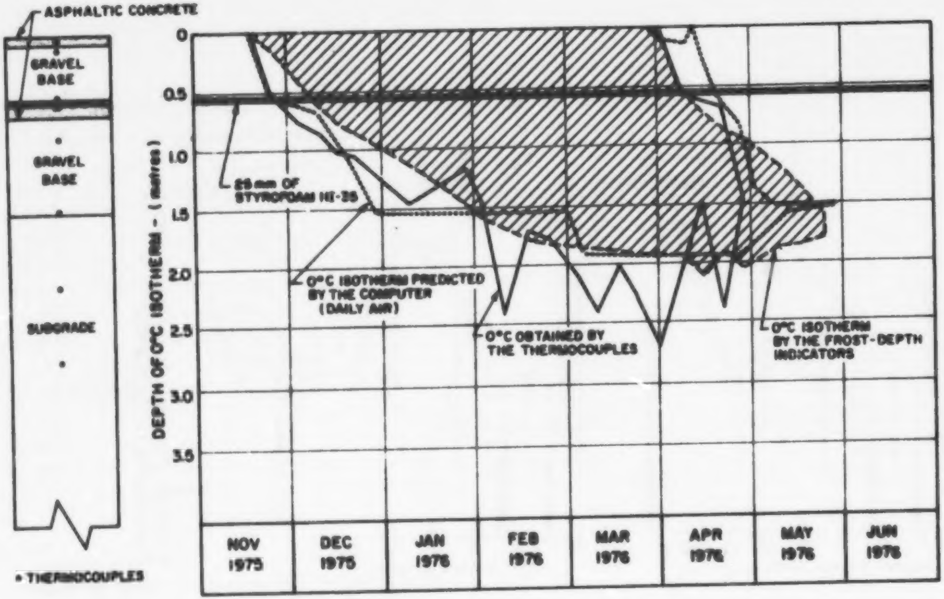
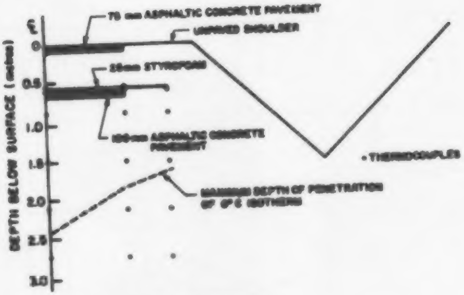
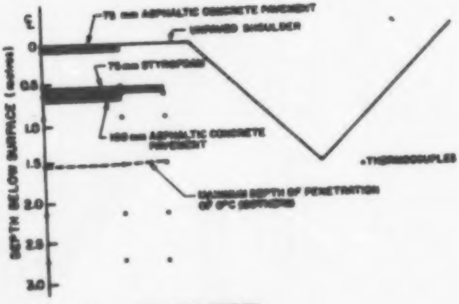


Figure 13/ Maximum Depths of Frost Penetration Recorded at the Centreline of the 25 mm Section During 1973-1974


75 mm ASPHALTIC CONCRETE PAVEMENT
UNPAVED SHOULDER
℄
0
0.5
1.0
1.5
2.0
2.5
3.0
DEPTH BELOW SURFACE (metres)
25mm STYROFOAM
100 mm ASPHALTIC CONCRETE PAVEMENT
• THERMOCOUPLES
MAXIMUM DEPTH OF PENETRATION OF 0° C ISOTHERM

(a) 25 mm INSULATED SECTION


75 mm ASPHALTIC CONCRETE PAVEMENT
UNPAVED SHOULDER
℄
0
0.5
1.0
1.5
2.0
2.5
3.0
DEPTH BELOW SURFACE (metres)
75mm STYROFOAM
100 mm ASPHALTIC CONCRETE PAVEMENT
• THERMOCOUPLES
MAXIMUM DEPTH OF PENETRATION OF 0°C ISOTHERM

(b) 75 mm INSULATED SECTION

Figure 14/ Transverse Cross Section of Maximum Frost Penetration (1975-1976)

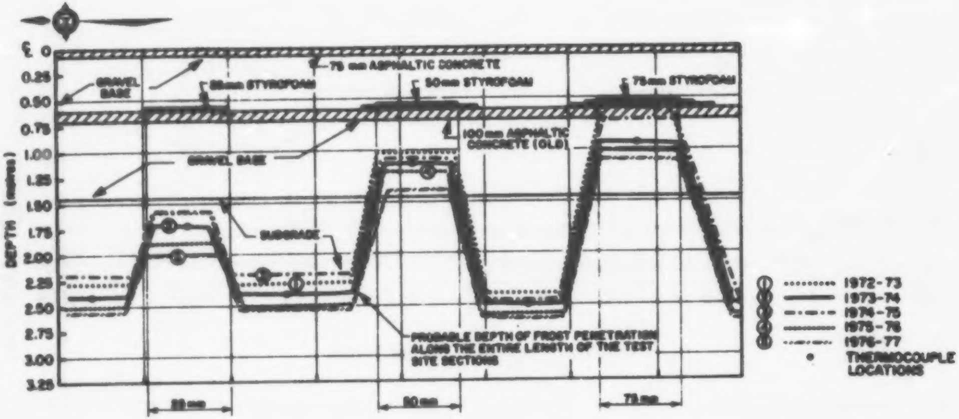


Figure 15/ Maximum Depths of Frost Penetration Throughout the Test Site During the Five Year Test Period

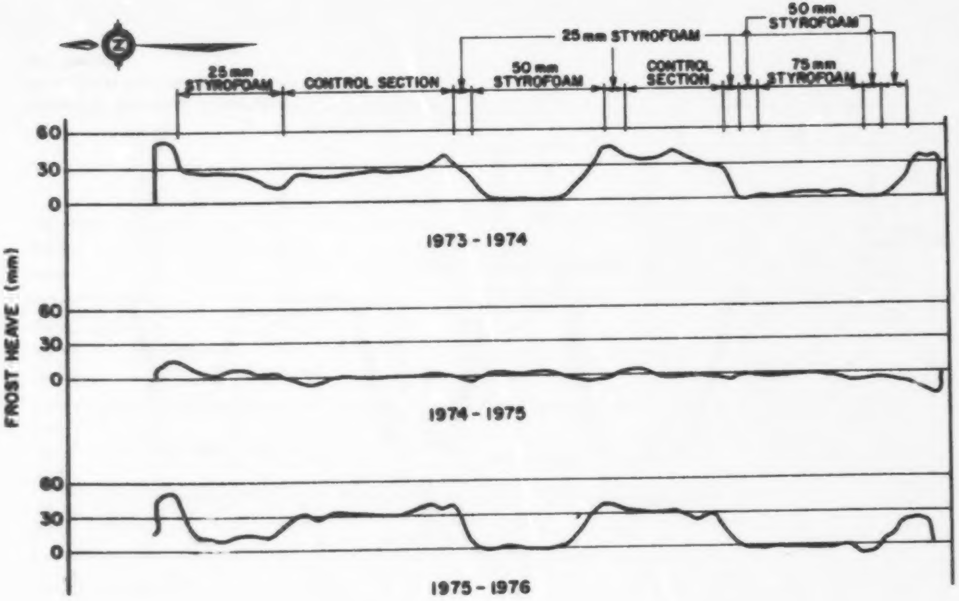


Figure 16/ Frost Heave Measurement Taken at the Pavement Centreline

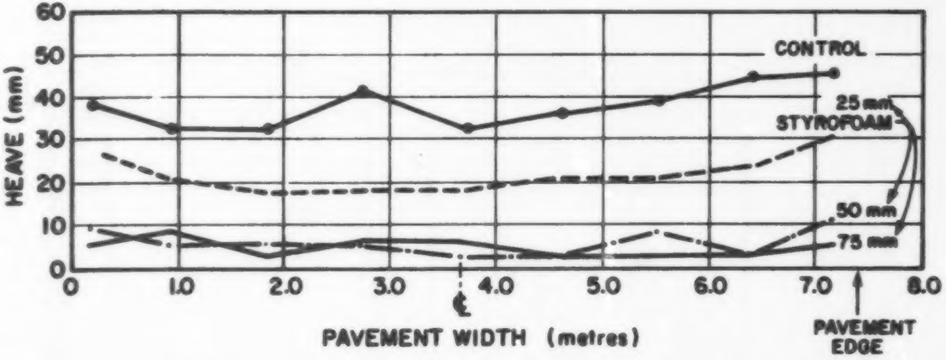


Figure 17/ Typical Heave Recorded in the Transverse Direction (1973-1974)

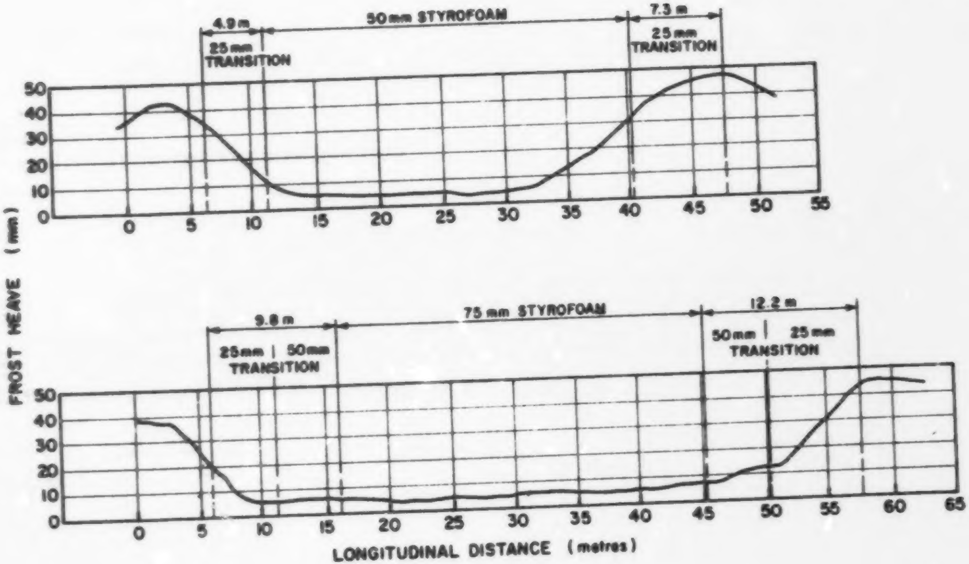


Figure 18/ Longitudinal Heaving Through the Transition Areas (1973-1974)

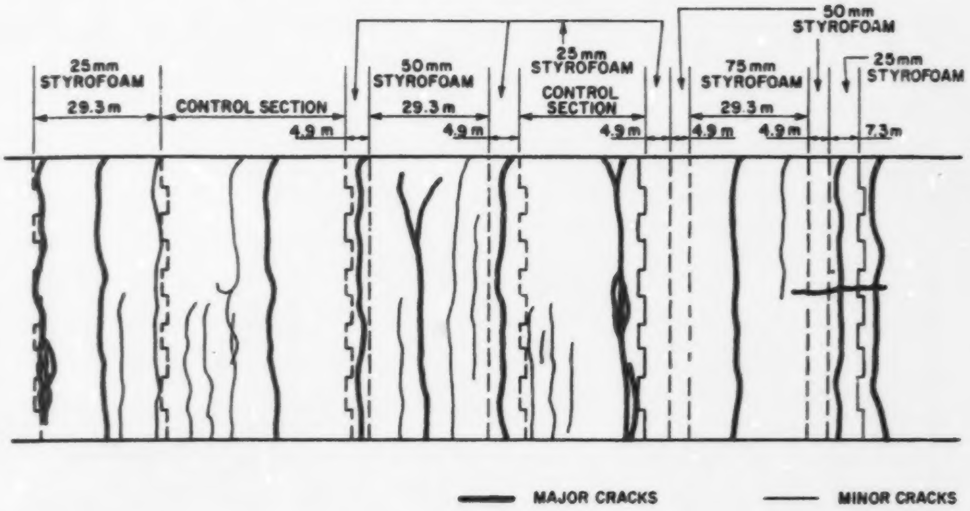

25mm STYROFOAM
29.3m
CONTROL SECTION
4.9 m
50mm STYROFOAM
29.3m
4.9 m
25mm STYROFOAM
CONTROL SECTION
4.9 m
50mm STYROFOAM
75mm STYROFOAM
29.3m
4.9 m
4.9 m
25mm STYROFOAM
7.3m
MAJOR CRACKS
MINOR CRACKS

Figure 19/ Pavement Cracking Pattern After the Five Year Test Period

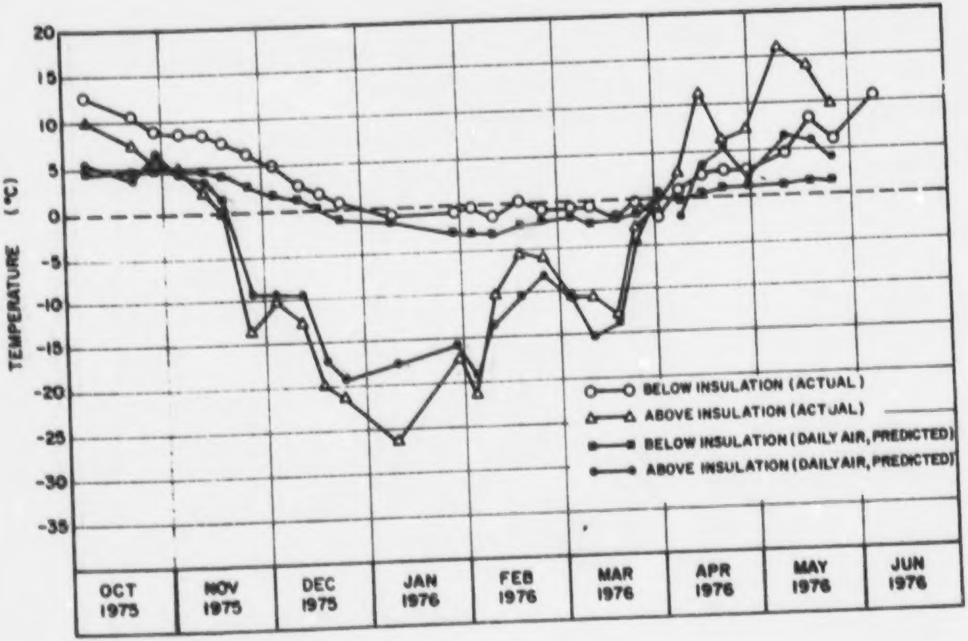


Figure 20/ Observed and Computed Temperatures Directly Above and Below the 75 mm STYROFOAM™ Layer at the Centreline

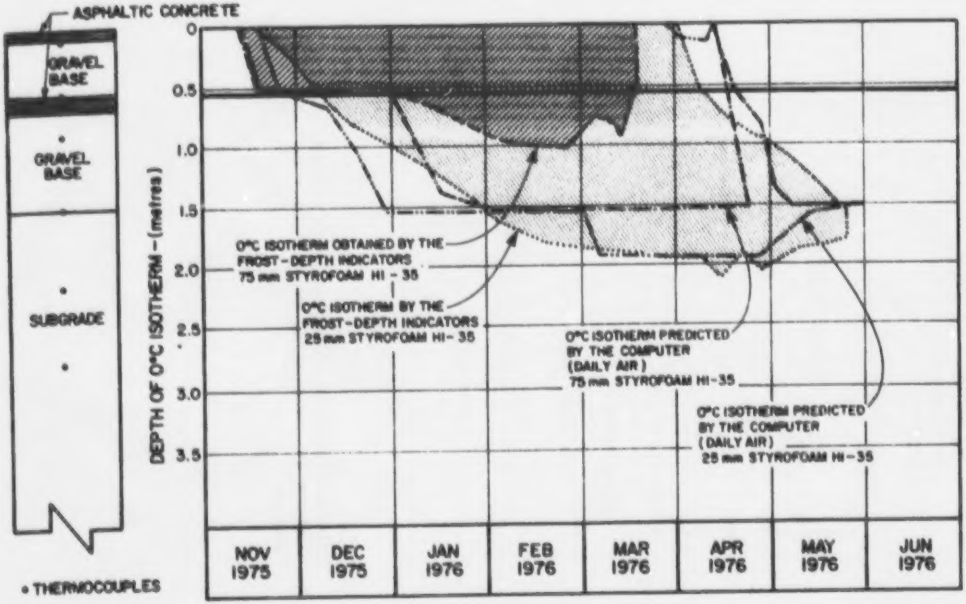


Figure 21/ Maximum Depths of Measured and Predicted Frost Penetration for the 25 mm and 75 mm Sections

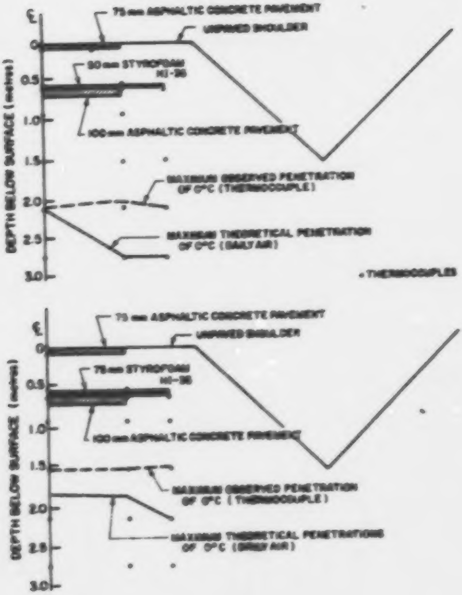


Figure 22/ Comparison of Measured and Predicted Maximum Frost Penetration in the Transverse Direction

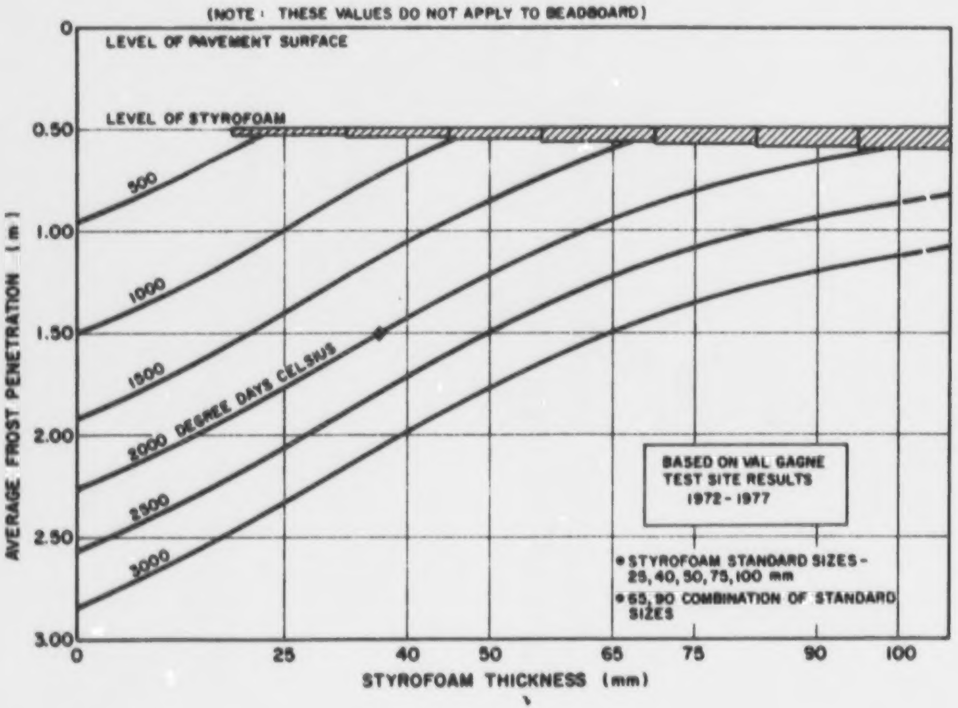


Figure 23/ Average Penetration of Frost Beneath STYROFOAM® in Paved Highways

Session 3

The Use of Cold-Mixed, Cold-Laid, Open-Graded Pavement to Minimize Transverse Cracking

W.H. MacDonald
District Municipality of Muskoka

D.F. Lynch and J.A. Cooper
Ontario Ministry of Transportation and Communications

Section 2

The Use of Cold-Mixed, Cold-Laid, Open-Graded Pavement to Minimize Transverse Cracking

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ABSTRACT

THE USE OF COLD-MIXED, COLD-LAID, OPEN-GRADED PAVEMENT TO MINIMIZE TRANSVERSE CRACKING

By

W. H. MacDonald

District Municipality of Muskoka

and

D. F. Lynch and J. A. Cooper

Ontario Ministry of Transportation and Communications

The advent of new equipment and the development of new sophisticated asphalt emulsions now permits the construction of cold-mixed, cold-laid open-graded pavements with some degree of success.

This paper outlines the success that the District Municipality of Muskoka in Northern Ontario has experienced with open-graded pavements. Particular reference is made to pavement performance with respect to minimizing low temperature cracking in an area where winter temperature reaches a low of -45°C; the average number of days of below freezing temperature is 145, with an average snowfall of 365 cm.

Over the past five years the District has found that open-graded pavement is much more flexible than any other type of pavement. The open-graded structure and thick asphalt films on the aggregate allow the pavement to conform to deformations without cracking. Resiliency and resistance to cold temperatures remain over a considerable length of time and provide added flexibility and self-healing to the pavement.

A brief outline of design, construction procedures, observations, and cost is included.

The District has achieved such good results that it is planning to continue the construction of open-graded pavements on an annual basis.

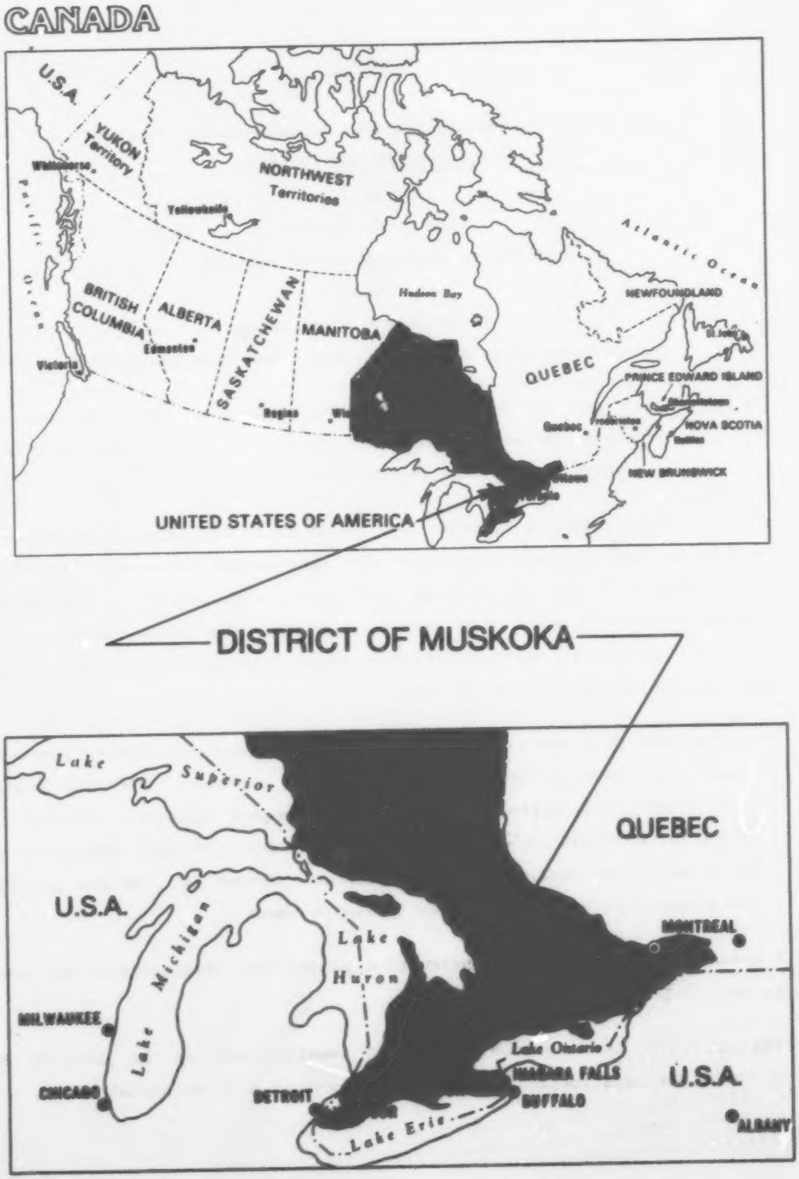
CANADA
U.S.A.
YUKON Territory
Whitehorse
NORTHWEST Territories
Yellowknife
Pacific Ocean
BRITISH COLUMBIA
ALBERTA
SASKATCHEWAN
MANITOBA
Edmonton
Victoria
Regina
Hudson Bay
Atlantic Ocean
NEWFOUNDLAND
QUEBEC
Quebec
PRINCE EDWARD ISLAND
NOVA SCOTIA
NEW BRUNSWICK
UNITED STATES OF AMERICA
DISTRICT OF MUSKOKA
Lake Superior
QUEBEC
U.S.A.
Lake Michigan
Lake Huron
MONTREAL
MILWAUKEE
CHICAGO
DETROIT
Lake Ontario
NIAGARA FALLS
BUFFALO
U.S.A.
Lake Erie
ALBANY

Figure 1/ Location of District of Muskoka

THE USE OF COLD-MIXED, COLD-LAID, OPEN-GRADED PAVEMENT, TO MINIMIZE TRANSVERSE CRACKING

By

W. H. MacDonald

District Municipality of Muskoka

and

D. F. Lynch and J. A. Cooper

Ontario Ministry of Transportation and Communications

INTRODUCTION

Paving engineers have long been concerned about transverse cracking of asphaltic concrete pavements. This cracking shortens the life of the pavement and reduces the riding quality.

A transverse crack is a crack which follows a course approximately at right angles to the centre line of the pavement. Transverse cracks are normally described as multiple, full, half or partial. Full transverse cracks tend to be regularly spaced along the length of the road, while half or partial cracks occur at shorter intermediate distances [2]. It is generally accepted that transverse cracking occurs when the temperature falls below the point at which the asphalt can no longer flow fast enough to relieve the stresses.

During the early spring there may be extended periods when a pavement experiences a number of freeze-thaw cycles when the temperature is above freezing during the day and below freezing during the night. Water entering the cracks during the day results in the formation of ice below the cracks during the night producing an upward lipping of the crack edges. This temporary condition results in a very poor riding quality pavement leading to complaints from the travelling public and especially from the trucking industry.

In very cold regions, thermal transverse cracks often appear where the entire pavement structure cracks to its full depth. This type of crack occurs when the entire pavement is frozen into a solid mass and rises above its normal summer level and the pavement contracts as the temperature drops; its strength is exceeded and a major crack develops. De-icing solutions can enter these cracks and cause localized thawing of the subbase and a pavement depression surrounding the crack [1].

Eventually other transverse cracks develop adjacent to the original crack and the pavement can develop an alligator cracking pattern. Any repairs attempted are generally unsatisfactory since the cracks reflect through in a very short time.

It has been shown that the number of transverse cracks increases with decrease in temperature and is proportional to the number of days below freezing during the winter months; that more open-graded pavements show less cracking; and that mixes of lower stiffness have less tendency to crack under thermal conditions [1]. It has also been noted that higher penetration asphalt cements result in mixtures of lower stiffness which results in reduced cracking [1].

A summary of recommendations and conclusions on the subject of transverse cracking is as follows [1].

(1) Transverse cracking is largely due to low temperature with greater cracking more evident in areas of low temperature over extended periods.

(2) In order to decrease cracking, mixes of lower stiffness should be used.

(3) The stiffness can best be controlled by the selection of softer asphalts or asphalts of lower temperature sensitivity.

This paper will describe the District of Muskoka's solution to the problem of transverse cracking.

DISTRICT OF MUSKOKA

Muskoka is located approximately 200 km north of Toronto in the province of Ontario. Muskoka is a year-round tourist region with numerous people on vacation in the summer taking advantage of the many lakes and waterways, enjoying swimming, water skiing and fishing and in the winter enjoying downhill and cross country skiing, snowmobiling, ice fishing and the beautiful snow-covered terrain. Summer temperature reaches a high of 35°C and mid-winter temperature reaches a low of -45°C. The average number of days of below freezing temperature is 145 with an average low temperature of -26°C, with an average snowfall of 365 cm and an average rainfall of 731 mm.

The District covers an area of 4700 km^2 and has a population of 38 700. This figure escalates to 220 000 during the summer months of July to September. The Average Automobile Daily Traffic Count (AADT) ranges from 2000 in the summer to 500 in the winter on the less-travelled roads, and from 17 000 in the summer to 2000 in the winter on the most heavily-travelled roads.

The area is part of the Laurentian peneplane. The peneplane surface rises gently from an elevation of 175 m above sea level at Georgian Bay to about 450 m near Huntsville. In the southern and western parts of the area the relief is relatively low, reaching only about 100 m in the Huntsville area. Although the general surface is a peneplane, the topography is quite irregular in detail and the area is dotted with many lakes separated by rocky ridges. Soil cover is generally sparse [5].

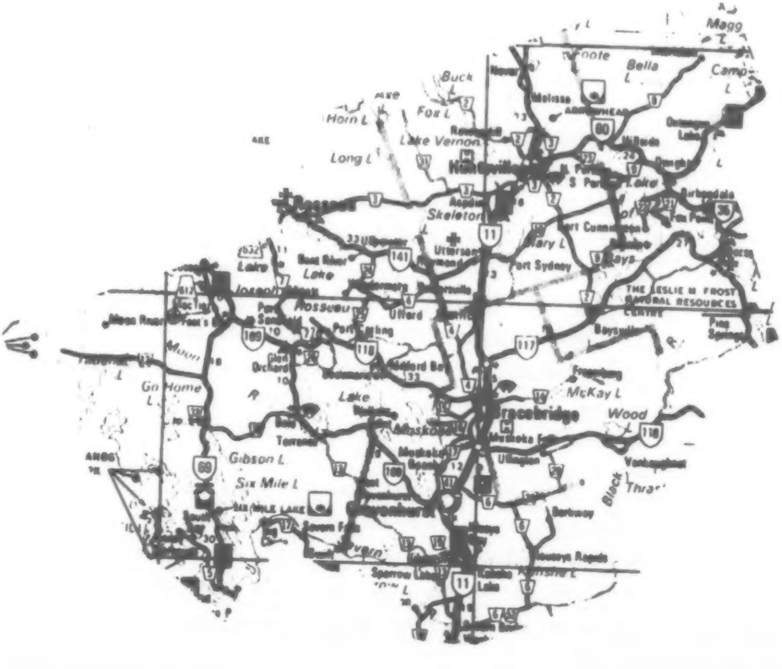
Bella L
Buck L
Fox L
Horn L
Lake Vernon
Long L
Huntsville
Skeleton L
Utterson
Port Sydney
Mary L
Rosseau
Joseph
Lake Muskoka
Bracebridge
Gravenhurst
McKay L
Wood
Gibson L
Six Mile L
Go Home L
Sparrow Lake
Kahshe Lake
THE LESLIE M. FROST NATURAL RESOURCES CENTRE
Black
11
60
117
118
141
169
35
69

Figure 2/ District of Muskoka

The area was covered in Pleistocene times by a thick ice sheet. During the withdrawal of the ice sheet to the north the area was covered by the waters of glacial Lake Algonquin. Extensive glacial deposits of sand and gravel exist in many parts of the area, e.g., Bracebridge, Parry Sound and Huntsville.

The bedrock formations of the area are of Precambrian age. The principal Precambrian rocks are granite and migmatitic gneisses, paragneiss and basic rocks. The bedrock is largely composed of veined, banded, and homogeneous pink and grey gneisses produced by injection and granitization of metamorphic gneisses of various types.

There are virtually no large tax-paying manufacturing industries in the District and most public revenues are derived from taxes collected from local residents and the hotel and resort owners and matching provincial grants or subsidies. This situation places severe restrictions on District expenditures of all kinds including the construction and maintenance of the road system.

District of Muskoka Road System

The road system in Muskoka is 550 km in length, composed of 470 km of bituminous surface and 80 km of gravel surface. The majority of the roads were built as summer access roads to cottages and resort areas, and to serve the lumbering industry, and were not constructed for year-round service. The roads were initially constructed on very weak or substandard bases using a minimum amount of gravel, and geometric standards were virtually non-existent. As the District began to open up and become more popular the more heavily-travelled roads began to receive some form of bituminous surface.

Bituminous surfaces included prime and single surface treatment, prime and double surface treatment, cold grader mix mulch using cut back asphalt and conventional hot-mix asphaltic concrete. The performance of these surfaces has not been satisfactory as they have experienced transverse cracking, frost heaving and structural failures associated with weak bases and low temperatures.

Experience with these types of pavements is detailed as follows.

Prime and Single Surface Treatment

The cost of this type of pavement was approximately $8700/km (1982 dollars). While the initial cost was low, performance was extremely poor, experiencing anywhere from 15% to 30% loss the first year with a major amount of patching required in order to keep the roads serviceable. This type of construction has been discontinued.

Prime and Double Surface Treatment

The cost of this type of pavement was approximately $11 200/km (1982 dollars). While the maintenance cost on this type of pavement was much lower than for prime and single surface treatments, its performance still left much to be desired. Its use was limited to roads where the base was well drained, structurally adequate and well compacted, and since such roads were few within the District this type of construction has been discontinued.

Road or Grader Mix Mulch

The cost of this type of pavement was approximately $16 000/km (1982 dollars). While the maintenance required on this type of pavement was minimal, it had other limitations. Construction of this type of pavement was a very slow process and totally dependent on dry, warm, sunny weather and required the use of equipment over an extended period of time. Such construction could only be carried out during the busy summer months when traffic was at a maximum, and caused great inconvenience to the travelling public. The District does not use this type of construction any longer.

Hot-Mix Asphaltic Concrete

The cost of this type of construction is currently $25 000/km. Initially the District attempted to use conventional hot-mix asphaltic concrete pavement over existing unimproved bases which had not been properly upgraded or reconstructed. Very poor performance was experienced due to severe transverse cracking, frost heave damage and structural failures. Because of this, the use of hot-mix is now limited to those roads where sufficient funds have been allocated to reconstruct the roadway to adequate structural and drainage standards.

Cold-Mixed, Cold-Laid, Open-Graded Pavement

Faced with poor performance and high maintenance costs of surface treatments, grader mix mulches and hot-mix pavements over existing weak road bases and with no prospects of obtaining sufficient funds to upgrade the roads system to suitable standards, the engineering department of the District was actively searching for an economical solution to their paving problem. They recognized the need for a low-cost, low-maintenance pavement with a high degree of flexibility, capable of minimizing cracking due to low temperatures and frost heave action. The District became aware of cold-mixed, open-graded pavement in the spring of 1975 when it was first introduced into Ontario.

The history of open-graded pavement construction dates back to 1968 in the Pacific North West of the United States in the State of Oregon. The Federal Highway Administration working in conjunction with the local U.S. Forest Service Bureau developed the design and construction of open-graded, emulsified asphalt pavement. Temperatures in Oregon are lower and winter weather conditions more severe than in Muskoka. The work has been confined mainly to rural logging and farm-to-market roads carrying a high amount of truck traffic. Performance of open-graded pavements in this area has been excellent, and the amount of work done increases each year.

After examining all available information and checking the concept in some detail with respect to materials and design, and examining newly-developed mobile mixing equipment such as the Midland Mix Paver designed primarily for mixing and laying open-graded pavement, the District decided to try the system. In September of 1975 the first two cold-mixed, cold-laid, open-graded pavement projects totalling 3.0 km in length were completed. Since that time 26 projects in all have been completed for a total of 62.0 km.

OPEN-GRADED PAVEMENT CONSTRUCTION IN MUSKOKA

General Description

Cold-mixed, cold-laid, open-graded pavements constructed in Muskoka consist of two components: a cationic mixing grade asphalt emulsion, and crushed stone with less than 10% passing the 4.75 seive and less than 2.0% passing the 75 um sieve. The air voids in the compacted pavement are controlled between 20% and 30%. The mixture is mixed and placed and compacted on the road with conventional equipment and construction procedures. A fine cover aggregate is spread on the pavement to prevent surface damage and pick-up by traffic. Table I shows the cost per kilometre and pavement thickness in millimetres of all projects completed.

TABLE 1

OPEN-GRADED PAVEMENT MUSKOKA

COST PER km

PAVEMENT THICKNESS mm

YEAR	75 mm	78 mm	89 mm	91 mm	92 mm	102 mm	108 mm	114 mm
1975			$ 9 708					$10 155
1976	$11 776		11 878	$12 477	$14 588			
	12 701							
	12 335							
1977	12 031		13 169		12 195			
	9 534		16 194		17 694			
	14 090							
1978	9 026				15 824			
					17 511			
1979	14 720			23 375		21 780		
	13 013	18 340						
1980	18 340							
	14 960							

Pavement Thickness

Pavement thickness has been based on local conditions, depth of existing gravel base, contour of the road, drainage and traffic volume. Normal minimum thickness has been 75 mm and maximum thickness has been 114 mm.

Aggregate

The aggregate used in Muskoka has varied depending on job location, and has been obtained from both limestone quarries and igneous gravel pits. The aggregate gradations selected are the same as those used in hot-mix pavements, so aggregate is readily and economically available from the commercial aggregate suppliers in the area.

The aggregates conformed to the gradation requirements shown in Table 2.

TABLE 2

GRADATION REQUIREMENTS FOR OPEN-GRADED AGGREGATE

Sieve Sizes	CL4 % Passing	CL8 % Passing
26.5 mm	-	100
19.0 mm	100	95-100
16.0 mm	96-100	65-90
13.2 mm	67-86	-
9.5 mm	29-52	20-55
6.7 mm	-	-
4.75 mm	0-10	0-10
2.36 mm	0-5	0-5
75 um	0-2	0-2

Gradation selection was dependent on the desired thickness of pavement required. When multiple lifts were placed, the first lift normally was constructed with CL8 aggregate.

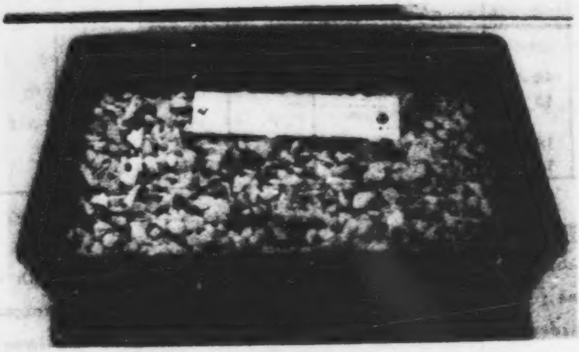

Figure 3/ Typical Open-Graded Aggregate

The aggregate used conformed to the physical properties shown in Table 3.

TABLE 3

PHYSICAL PROPERTIES FOR OPEN-GRADED AGGREGATES

	CL4	CL8
Los Angeles Abrasion Loss % Maximum	35	35
Magnesium Sulphate Soundness 5 Cycles Loss % Maximum	12	15
Absorption by Weight % Maximum	2.0	2.0
Petrographic Number Maximum	160	160
Loss by Washing Passing 75 um Sieve % Maximum	2.0	2.0
Flat or Elongated Pieces % Maximum	20	20
% Crushed Minimum (With 1 or More Fractured Faces)	90	90

Emulsified Asphalt

The emulsified asphalt used for the work was a CMS 2 grade and conformed to the requirements shown in Table 4.

TABLE 4

GENERAL REQUIRMENTS FOR CMS2 EMULSIFIED ASPHALT

REQUIREMENTS	Min.	Max.
VISCOSITY, SAYBOLT FUROL SECONDS At 50°C	50	400
RESIDUE BY DISTILLATION, %	65	-
SETTLEMENT, % 5 Days	-	5
OIL PORTION OF DISTILLATE, % by Volume	-	12
SIEVE TEST, %	-	0.10
COATING TEST, %	80	-
PARTICLE CHARGE	Positive	
TESTS ON RESIDUE Penetration at 25°C, dmm	100	250
Ductility at 25°C, cm	60	-
Solubility in Trichlorethylene, %	97.5	-

Cover Aggregate

The cover aggregate used conformed to requirements shown in Table 5.

TABLE 5

GRADATION REQUIREMENTS FOR COVER AGGREGATE

Sieve Sizes	% Passing
6.7 mm	100
4.75 mm	85-100
2.36 mm	55-100
1.18 mm	30-90
600 um	15-70
300 um	5-40
150 um	0-15
75 um	0-7

This material consisted of either sand or quarried screenings produced during the manufacture of CL4 or CL8 aggregate.

Testing

Prior to the commencement of the work the contractor was required to carry out laboratory tests to establish the mix design.

The tests required were:

(A) Coating Test for Open-Graded Aggregate, and

(B) Run-Off Test for Open-Graded Mix.

For a complete description of these tests see Appendix A and B.

These tests indicate the most suitable aggregate/emulsion combination for the work which provides good coating of the aggregate, workability of the mix without stripping of asphalt from the aggregate and acceptable curing time to resist wash off should rain fall within one hour after placement.

The residual asphalt content conformed to the limits shown in Table 6.

TABLE 6

RESIDUAL ASPHALT CONTENT LIMITS

Type of Mix	Asphalt Emulsion Content in Kilograms per tonne	Minimum Residual Asphalt Content by Mass	Maximum Residual Asphalt Content by Mass
CL4 OPEN-GRADED MIX Surface, Levelling or Binder Course	61-107	4.0%	7.0%
CL6 OPEN-GRADED MIX Levelling or Binder Course	55-77	3.5%	5.0%

Surface Preparations

Open-graded mix has been placed over existing granular surfaces and over a variety of old bituminous surfaces. Granular surfaces were reshaped, graded and rolled with a minimum 7 t tandem steel-wheel roller. Old bituminous surfaces were swept clean of dirt and debris to ensure good bond, and any pot-holes were patched.

Mixing and Laying of Open-Graded Mix

All open-graded pavement constructed in Muskoka has been constructed using a Midland Mix Paver which is a self-propelled mobile plant designed and built to mix and place cold bituminous mixtures in one continuous operation. It consists of a folding hopper into which a dump truck discharges the aggregate, a 1.2 m wide constant speed conveyor belt with an aggregate strike off door for delivery of the aggregate to the twin shaft pugmill mixer where the emulsion is sprayed by two overhead spray bars and mixed with the aggregate. The emulsion is controlled by means of a variable speed pump and is metered into the mixer. The mixed material is then fed to the vibrating screed through two augers. Pavement thickness is controlled by the height of the floating screed which is fully adjustable from both ends and it can also be adjusted for crown control at the centre if necessary. The operation of the screed is identical to that of a hot-mix paver. The screed can also be hydraulically expanded when extra width is required.

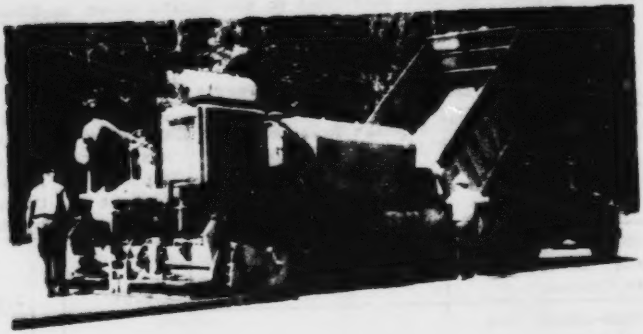

Figure 4/ Midland Mix Paver

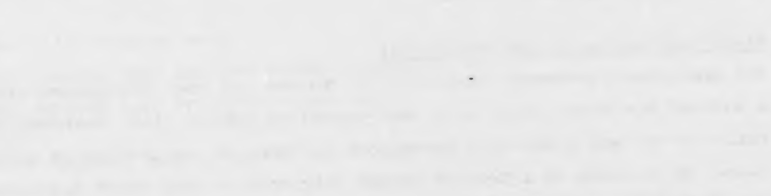

Figure 5/ Diagram of Midland Mix Paver

Open-graded pavement is usually weather proof as soon as it leaves the mixer. The emulsion is formulated so that the break of the emulsion, that is, the moment at which the asphalt coagulates and plates out on the aggregate, is almost instantaneous during mixing. The property of the asphalt residue is altered so as to prolong the final setting time and allow the mix to be conveyed through the mixer and laid to proper contour and depth.

The mix composition was required to conform to the following tolerances:

- residual asphalt content ± 0.3 %, and
- aggregate retained on the 4.75 mm sieve ± 2.0%.

Compaction of Mix and Spreading of Cover Aggregate

Initial Rolling

Initial rolling was carried out using a tandem steel-wheel roller (7.0 t minimum). Rolling commenced once the surface breaking of the emulsion occurred and the mix was able to support the roller without shoving, and was limited to one complete pass.

Spreading of Cover Aggregate

After the initial rolling, the cover aggregate was spread uniformly at about 5 kg/m^2 over the surface. The cover aggregate was spread using either a self-propelled aggregate spreader, or an acceptable tail-gate spreader capable of uniform application without displacement of the pavement. Spinner-type spreaders were not allowed because they do not spread uniformly.

The purpose of the cover aggregate was to prevent surface damage and pick-up by traffic. In areas of heavy traffic a heavier application of cover aggregate was used. Traffic was not allowed on the pavement until the cover aggregate was spread and rolled.

Intermediate Rolling

After the application of the cover aggregate, one pass was made with a 7.0 t rubber-tired roller to ensure that any areas bridged by the steel-wheel roller were compacted.

Final Rolling

Final rolling was done using a 7.0 t tandem steel-wheel roller to provide a flat, smooth, rut-free surface.

Payment for Construction of Open-Graded Pavement

All open-graded pavement was constructed on a contract basis. Each contract contained four tendered items for supplying all labour equipment and materials for completion of the work, namely:

- supply of asphalt emulsion,
- supply mix and place CL4 aggregate,
- supply mix and place CL8 aggregate, and
- supply and apply cover aggregate.

The emulsion was paid for by the litre and all the aggregates by the tonne.

Construction Observations

For best results, the work should not be started unless the ambient temperature is 10°C and rising, to ensure full cure. Low temperatures retard curing and some rutting might result. In theory, no work should be started after September 15th in Muskoka because of short curing days and low night-time temperatures. However, work has been done successfully in Muskoka at the end of October when night-time temperatures might be as low as 0°C. Paving at such low temperatures is not recommended.

Difficulties were encountered when the moisture content of the aggregate was greater than 2.0%. The moisture caused some of the emulsion to run-off and reduced the asphalt residue coating on the aggregate. Slow curing also resulted when excess moisture was present (Appendix C) [4].

It was noticed that if more than 10.0% was passing the 4.75 sieve or 2.0% passing the 75 um sieve, the surface area to be coated was increased and more difficult to coat. Insufficient coating or more rapid break of the emulsion occurred causing mixing, laying or compaction problems. When the emulsion content was increased, the coating was improved but it resulted in reduced stability and increased emulsion run-off as well as escalated costs. To try to improve coating by extending the mixing period will cause the asphalt to strip from the aggregate or break and set up prematurely in the mixer or on the road (Appendix C) [4].

Experience has shown that open-graded pavement will usually resist water damage during construction; nonetheless, no work should be started when rain is imminent. Rainfall will extend the length of the curing period and if rain occurs traffic should be kept off the pavement until the mix has cured and the necessary compaction has taken place; otherwise wheel rutting will occur.

Faster curing will occur if the mix is placed in two or three layers rather than one thick layer.

Over the years three types of rollers have been used, and it has been found that compaction can be achieved using any type of roller; however, tandem steel-wheel rollers are preferred. Rubber-tired rollers have a tendency to pick up material from the mat and leave ridges in the finished pavement. Vibratory steel-wheel rollers may be used but extreme caution should be exercised because overuse will overcompact the mix, reduce the void content, and cause the emulsified asphalt to migrate from the aggregate to the sub-base.

PERFORMANCE

All of the open-graded pavements constructed in the District of Muskoka since 1975 are performing well at this time. There has not been a single pavement failure nor have any pavement maintenance dollars been spent on any of the roads to date. All of these pavements were placed on unimproved existing roads.

A Case Study - Muskoka Road Number 2

Muskoka Road Number 2 provided a unique opportunity to directly compare the performance of open-graded cold-mix with conventional hot-mix and recycled hot-mix. The section of Road 2 in question is located near Huntsville and runs from King's Highway Number 11 westerly to Ravenscliffe. The existing gravel surface was not rebuilt before paving; it was, however, reshaped and compacted. Sub-grade conditions are similar under all three connecting sections. A survey of these sections was carried out on June 15, 1982 to determine frequency of transverse cracking, to evaluate skid resistance, and to make an assessment of frost heave damage.

Results of the transverse cracking survey are shown in Table 7.

Typical Transverse Cracks, Muskoka Rd. 2

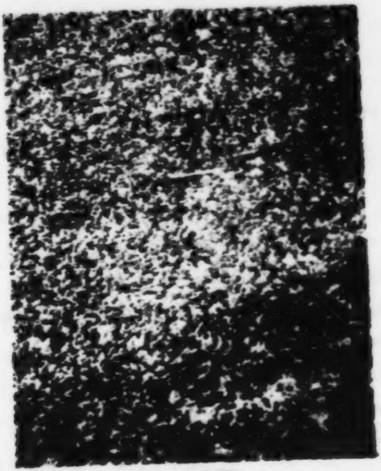

Figure 6/ Open-Graded Mix

Figure 7/ Conventional Hot-Mix

Figure 8/ Recycled Hot-Mix

TABLE 7

TRANSVERSE CRACK SURVEY RESULTS

DISTRICT ROAD NUMBER 2	TYPE OF PAVEMENT	YEAR CONSTRUCTED	COST/km	LINEAR METRES OF TRANSVERSE CRACKS PER km OF PAVEMENT	AVERAGE SPACING BETWEEN CRACKS IN METRES
Section A	Open-Graded Mix	1979	$ 13 013	33	156
Section B	Conventional Hot-Mix	1980	$ 25 645	158	38
Section C	Recycled Hot-Mix	1981	$ 23 441	180	33

On examination of Table 7 it is clear that open-graded pavement dramatically minimizes transverse cracking when compared to hot-mix; the results are even more impressive when one considers that the open-graded section is the oldest since it has been observed that the frequency of transverse cracking increases with age. It is also evident that open-graded mix enjoys a considerable cost advantage when compared to hot-mix.

Skid measurements were taken using the Ontario Ministry of Transportation and Communications skid trailer conforming to ASTM Standard E274 as shown in Figure 9 [4].

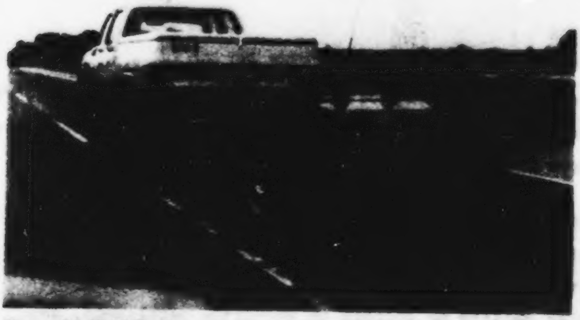

Figure 9/ Skid Trailer

Results of the 80 tests conducted at approximately 50 km/h are shown in Appendix D.

A summary of the skid resistance tests is shown in Table 8.

TABLE 8

SKID RESISTANCE TEST RESULTS

Section	Pavement Type	Westbound Lane SN 50 km/h (Range)	Eastbound Lane SN 50 km/h (Range)
Section A	Open-Graded Mix	46.4 (44-52)	42.1 (38-48)
Section B	Conventional Hot-Mix	43.8 (37-45)	41.8 (33-51)
Section C	Recycled Hot-Mix	51.2 (41-57)	43.3 (38-50)

The above averages show that all these sections have excellent skid resistance in the initial stage of their life. The open-graded mix, which has a harsh macrotexture, would be more desirable for high speed (70 km/h) plus traffic.

Visual observations of areas where frost heaves have occurred indicate no break up of the open-graded pavement, and where cracks were visible in the spring, these cracks have resealed. In both the conventional hot-mix and recycled hot-mix sections the frost heave areas have cracked badly and are suffering from pavement distress and break up, and require attention.

CONCLUSIONS

Muskoka has concluded on the basis of all evidence to date that the durability and life expectancy of cold-mixed, cold-laid, open-graded pavement given the same sub-base, weather, and traffic conditions exceeds that of other types of surfacing.

Open-graded mix outperformed hot-mixes in the following areas:

(1) better resistance to transverse cracking,

(2) better capacity to accommodate frost heaves,

(3) better ability to conform to sub-grade settlement, and

(4) self-healing properties when cracking does occur.

There is a considerable cost saving in the price per tonne laid of open-graded mix vs. hot-mix. Also, there is an additional cost saving when open-graded pavement is used as there is no need for costly reconstruction of the road base with large amounts of gravel prior to paving, as is normally required prior to laying hot-mix.

Results achieved by the District of Muskoka using cold-mixed, cold-laid, open-graded pavement have been very satisfactory and the District is planning to continue to construct this type of pavement on an annual basis.

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APPENDIX A

COATING TEST FOR OPEN-GRADED AGGREGATE

This method evaluates the ability of an asphalt emulsion to coat an open-graded aggregate and resist the washing action of water after the completion of mixing.

(1) Weigh 500 g of dried aggregate (100% passing 13.2 mm and retained on 4.75 mm) at 25°C into a metal mixing bowl.
(2) Moisten the aggregate with the minimum amount of tap water required to make the aggregate in a surface damp condition. Normally, this is just enough water to darken the aggregate with no evidence of free water.
(3) Add 6% emulsion at 25°C (based on the dry aggregate weight) and mix with a metal spatula. Note the time when the mix stiffens. Do not mix after the mix is 100% coated or longer than three minutes.
(4) Remove approximately one-half of the mixture. Gently rinse the remainder with water until the water runs clear. Drain off all water.
(5) Allow the rinsed and unrinsed samples to air dry on felt paper towel at room temperature until completely dry.
(6) When dry, visually rate the % coating of each.
(7) Test higher and lower emulsion levels depending on the % coating obtained.

INTERPRETATION OF RESULTS

- Mixes which strip or stiffen excessively are unsatisfactory.
- 90% coating is considered excellent.
- 80-90% coating is considered fair to good.
- Less than 80% coating is considered poor.
- Should the retained coating be less than 80%, formulation adjustments within the specification limits for the asphalt emulsion shall be attempted and tests repeated to obtain a suitable emulsion.
- If a minimum of 80% coating cannot then be achieved, the aggregate shall be rejected.

APPENDIX B

RUN OFF TEST FOR OPEN-GRADED MIX

This method is used to evaluate the asphalt run off from the open-graded aggregate.

(1) Weigh 500 g of dried aggregate (100% passing 13.2 mm and retained on 4.75 mm) at 25°C into a metal mixing bowl.

(2) Moisten the aggregate with the minimum amount of tap water required to make the aggregate in a surface damp condition. Normally, this is just enough water to darken the aggregate with no evidence of free water.

(3) Add at 25°C predetermined amount of emulsion to obtain desired coatability and mix with metal spatula. Do not mix after the mix is 100% coated or longer than three minutes.

(4) Pour or spoon all of the mixture into 850 um mesh wire screen funnel which has been positioned over a 600 mL beaker.

(5) Allow the mix to drain at room temperature for 30 minutes.

(6) Place the beaker containing the run off in a 110°C oven and dry to constant weight. Determine this final weight.

(7) Compute % run off = $\frac{\text{final weight - tared weight}}{\text{dry aggregate weight}}$ x 100

INTERPRETATION OF RESULTS

- The maximum amount of emulsion that the aggregate can retain is desirable.
- Mixes are considered acceptable if their run off is less than 0.5% residual asphalt by weight of dry aggregate.

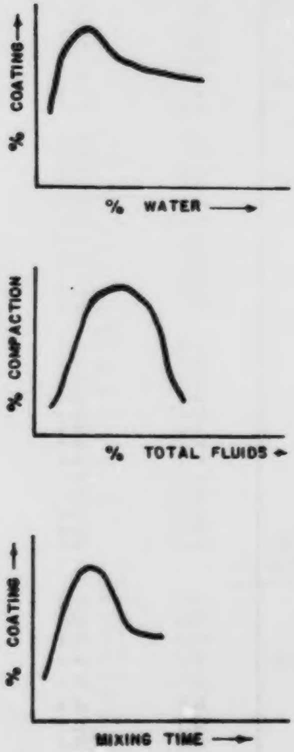

Figure C-1/ Open-Graded Mix Characteristics

Hwy No. MUSKOKA RD. 2

Location 1 KM NORTH CF HUNTSVILLE - STARTED APPROXIMATELY
10 KM WEST OF HWY. 11

Date 82 06 14

Direction EASTBOUND

Lane No. D.L. ONLY

WEST

OFFSET	SPEED	SN	INTERSECTION
0.00			Open-Graded Cold-Mix Starts
0.11	57.3	42.0	
0.21	59.5	37.7	
0.30	52.8	40.1	
0.37	48.7	45.3	
0.46	45.8	42.5	$\overline{SN}_{50}$ = 42.1
0.53	45.8	37.6	
0.60	47.9	42.7	
0.68	54.7	42.3	
0.81	52.0	40.8	
0.90	44.8	48.2	
0.94			Conventional Hot-Mix Starts
1.10	45.0	48.0	
1.21	44.3	41.6	
1.29	46.8	46.6	
1.36	43.3	50.5	
1.47	43.1	42.5	
1.58	52.1	37.1	
1.68	51.8	40.9	
1.75	45.1	49.4	
1.80	53.5	37.6	
1.83	52.8	38.1	
1.92	53.7	37.9	
			$\overline{SN}_{50}$ = 41.3
2.05	51.8	40.5	
2.14	49.1	41.4	
2.15	50.7	42.4	
2.27	54.0	36.8	
2.35	56.6	42.3	
2.38	51.5	40.5	
2.46	50.2	45.9	
2.56	49.5	45.0	
2.57	48.9	40.4	
2.66	53.1	37.0	
7.68	50.3	33.3	
2.74	48.0	33.0	
2.79	53.6	36.1	
2.85			Ashley Rd.
2.87			Start of Recycled Hot-Mix
2.91	53.5	44.9	
3.12	55.6	50.2	
6.23	57.0	41.8	
4.34	57.1	48.8	
3.44	53.7	47.6	
3.54	52.6	45.6	$\overline{SN}_{50}$ = 43.8
3.66	51.3	42.7	
3.78	55.3	43.2	
3.90	54.4	41.4	
3.97	53.6	41.0	
4.05	52.3	37.9	
4.13	49.0	42.6	
4.21			End of Recycled Hot-Mix

Figure D-1/ Skid Resistance Data

HIGHWAY NO: REGION OF MUSKOKA ROAD 2

LOCATION: 1 KM NORTH OF HUNTSVILLE - STARTED APPROXIMATELY
10 KM WEST OF HWY. 11

DATE: 82 06 14

DIRECTION: WESTBOUND
LANE NO: 1 LANE ONLY

↓
WEST

OFFSET	SPEED	SN	INTERSECTION
0.00			Start of Open-Graded Cold-Mix
0.08	44.7	47.8	
0.17	42.2	52.3	
0.27	38.6	47.7	
0.44	47.3	43.9	
			$\overline{SN}_{50}$ = 46.4
0.62	43.0	50.3	
0.77	55.0	46.2	
0.89	53.6	44.6	
0.94			Conventional Hot-Mix Starts
1.04	62.9	41.0	
1.17	64.8	36.7	
1.29	55.7	36.7	
1.38	50.0	43.2	
1.55	61.7	40.1	
1.66	54.7	40.2	$\overline{SN}_{50}$ = 43.8
1.76	55.6	43.5	
1.91	54.4	43.1	
2.15	54.7	44.5	
2.26	53.1	40.5	
2.37	53.8	43.7	
2.49	52.3	45.1	
2.67	50.8	42.0	
2.77	52.1	44.9	
2.85			Ashley Road
2.87			Start of Recycled Hot-Mix
2.95	52.5	41.1	
3.06	54.3	48.9	
3.17	53.3	51.4	
3.27	54.4	50.9	
3.37	54.0	49.8	$\overline{SN}_{50}$ = 51.2
3.47	51.3	52.4	
3.61	57.8	49.4	
3.76	46.7	57.2	
3.87	50.6	50.9	
3.98	50.8	55.3	
4.09	49.6	49.8	
4.17	46.2	51.6	
4.21			End of Recycled Hot-Mix

Figure D-2/ Skid Resistance Data

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Paving in Cold Areas

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17. Abstract

The joint Canada/Japan Paving in Cold Areas Mini Workshop held in Vancouver between October 18 - 22, 1982 brought together Canadian and Japanese experts in the areas of pavement research to discuss the particular problems of paving in cold areas. The workshop consisted of six sessions at which twenty-three papers were presented and one panel discussion took place. The six major topic areas covered in the workshop were: low temperature cracking, materials and construction to overcome freezing problems in soils, frost penetration and the prevention of frost heaving, the prevention and cure of reflection cracking, characteristics of paving materials, and additives in asphalts.

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17. Résumé

Le mini-atelier conjoint Canada-Japon intitulé Les revêtements dans les climats froids tenu à Vancouver entre les 18 et 22 octobre 1982 a réuni un groupe d'experts canadiens et japonais dans le domaine de la recherche sur les revêtements afin de discuter les problèmes particuliers des revêtements dans les climats froids. L'atelier a compris six séances, durant lesquelles on a présenté vingt-trois communications techniques, et une session de discussion par des panelistes. Les six sujets généraux discutés pendant l'atelier furent: la fissuration à basse température, les matériaux de construction utilisés afin de combattre les problèmes de gel dans les sols, la pénétration du gel et les mesures préventives contre le gonflement dû au gel, les mesures préventives contre la fissuration due à la réflexion, les caractéristiques des matériaux utilisés pour les revêtements, et les additifs utilisés dans les asphaltes.

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Paving in Cold Areas

Mini-Workshop

October 18-22, 1982

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FOREWORD

At the 4th Canada/Japan Consultations on Science and Technology held in Tokyo, June 3-6, 1980, it was agreed that one of the topics where information exchange and collaborative activity would be of mutual benefit is Paving in Cold Areas.

The contact points named to foster this activity are:-

William A. Phang
Head, Pavement Research
Ontario Ministry of
Transportation and
Communications

and

Takashi Iijima
Head, Pavement Division
Public Works Research
Institute
Japan Ministry of
Construction

Information exchange was initiated during a visit to Japan by W.A. Phang and I.F. Gilbert in November 1980, and was continued during reciprocal visits to Canada by T. Sugawara, S. Matsuno, H. Kubo, and H. Ishii at various times in 1981. This Mini-Workshop is intended to further extend the depth and scope of the exchange by involving larger numbers of Canadian and Japanese engineers.

The first announcement and call for papers for the Mini-Workshop was issued in November 1981. The response from both Canadian and Japanese engineers is most gratifying.

The papers presented in the Proceedings are organized in the same way as in the program and consist of the following:-

Session I -Low Temperature Cracking
Session II -Materials and Construction to Overcome Problems of Freezing in Soils
Session III-Frost Penetration and Prevention of Frost Heave in Pavements
Session IV -Reflection Cracking, Prevention and Cure
Session V -Paving Materials
Session VI -Additive Research

Continued next page.

The meeting will be conducted in English. However, adequate time is allowed for English/Japanese translations and vice versa.

Each paper will be presented by the author or his designated representative. Questions, and discussions between authors and participants, are encouraged throughout the sessions.

Authors were asked to follow a few simple instructions in preparing their manuscripts, and the result presents a pleasing uniformity in appearance. I thank the authors for the time, effort, and work which they put into the preparation of their papers, and trust they will in turn benefit from their participation in this Mini-Workshop.

I thank Mr. Takashi Iijima, PWRI, Japan Ministry of Construction for organizing the whole of the Japanese response to the Mini-Workshop. I thank Mr. Don F. Martin, Director of Paving, B.C. Ministry of Transportation and Highways, for overseeing the local arrangements for the Mini-Workshop and the Field Trip. I thank the Ontario Ministry of Transportation and Communications for their support, particularly Lorraine Sosney and members of the Editorial Staff of the Policy Planning and Research Branch. And lastly, I thank Mr. Peter Eggleton and members of his staff at Transport Canada, Transportation Research Centre, for their assistance and for their financial support.

William A. Phang, Chairman

Paving in Cold Areas Mini-Workshop,
Research and Development Branch,
Ministry of Transportation and Communications,
1201 Wilson Avenue
Downsview, Ontario, Canada M3M 1J8.

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Preventing Reflection Cracking

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ABSTRACT

As new pavements age, cracking develops almost without exception. The type, frequency and severity of cracking is related to various factors which include design parameters, materials and environment. The serviceability of a roadway may, in time, be adversely affected, particularly by the environment, as a result of such cracking. Riding comfort declines, user costs rise, driver safety may be jeopardized and indeed, the structural integrity of the highway could be in peril if some form of rehabilitation is not carried out.

One of the most common methods of restoring the serviceability level of a highway is to apply an asphalt concrete overlay. Unfortunately, the formation of reflection cracks often results in a premature decline in the design life of the overlay. This problem is compounded in seasonal frost areas due to the freezing of moisture which enters the pavement structure through these cracks.

In this paper, the authors illustrate briefly how reflection cracking in Ontario is influenced by the factors mentioned above. A review is carried out of various methodologies and techniques which the Ministry has employed in recent years, to minimize reflection cracking in overlays. Details are given on the use of modified asphalt mixes as binder courses. These mixes are open graded in nature and have been placed over flexible and rigid pavements. Trials have been implemented utilizing variable overlay depths on different types of pavements. The Ministry's experience with geotextiles used for overlay reinforcement purposes, is also summarized.

Conclusions are drawn about the most effective and economically viable treatments that have been tried. Recommendations are given with respect to the use of new methodologies, techniques and materials by M.T.C. in future efforts to prolong pavement life and protect the investment made in Ontario's highway system.

PREVENTING REFLECTION CRACKING

by

J.B. MacMaster and W.E. Blum

Ontario Ministry of Transportation and Communications

INTRODUCTION

The successful performance of a pavement is determined primarily by the degree of service or level of serviceability provided to the driving public. This level of serviceability is at a maximum when the pavement is new, and it often stays that way for a number of years following construction. In time, however, deficiencies develop in the pavement, manifested most clearly by visible distresses in the driving surface. These distresses include various types of cracking. As cracks develop in frequency and severity, the level of serviceability begins to drop (Figure 1), leading to potential problems which must be addressed by the highway engineer.

Decline in Riding Comfort

A drop in riding comfort is the most obvious sign of roadway deterioration to a motorist. In its early stage, cracking itself may not contribute directly to a decline in riding comfort, however, when combined with environmental factors such as moisture and freezing temperatures, deterioration of the pavement is accelerated and the roughness of the ride is accentuated.

Pavement Life is Shortened

It is widely accepted that excess moisture trapped within a pavement structure is very damaging to the structural integrity of the roadway. New pavements are relatively impervious to surface moisture. As cracks develop, moisture has more direct access to the base and subbase layers. High pore pressures are created under the dynamic loading of traffic and frictional forces between the granular particles are overcome. This leads to a reduction in the bearing capacity of the base which in turn creates an overstressing of the wearing course (1). Failure to remedy the situation results in accelerated deterioration of the pavement and a need for rehabilitation before the design life is achieved.

User Costs Increase

It is somewhat difficult to assign numerical values to user costs, as this aspect of transportation economics is often ignored (2). User costs, nevertheless, are related to the performance history of a pavement. A high level of roughness over a prolonged period will result in additional expenditures being made by the road user. Vehicle operating costs rise due to increased fuel consumption resulting from forced changes in speed, accelerated wear on tires and more frequent replacement of parts. There is a cost associated with the increased time the motorist must spend in travelling and another indirect, but real cost is related to the irritation and frustration of the driver when faced with the above problems.

Decrease in Driver Safety

Rough pavements can lead to a higher risk of accidents, especially if pavement deterioration reaches the stage at which distortion and actual disintegration of the wearing course is prominent (Figure 2). Injury, death and property damage all have inherent costs which must be borne by the public in general.

RELATIONSHIP OF PAVEMENT CRACKING TO EXTERNAL FACTORS

Cracking of pavements, be they flexible or rigid, is related to various factors, the most significant of which can be grouped into general categories of environment, materials and design.

Environment

In the severe climatic conditions of Canada, highway pavements are subjected to high stresses due to dramatic temperature changes. Thermal cracking of asphalt concrete pavements is a major concern, particularly where relatively thin wearing courses (75 mm or less) are constructed. These two factors apply particularly in northern Ontario.

During the winter season, the Ministry uses salt as a deicing agent to maintain clear driving surfaces. This practice, however, contributes to the supply of moisture which can enter the pavement structure during the day. This water freezes at night and causes the pavement to heave. Repeated freeze-thaw cycles eventually lead to overstressing of the asphalt concrete and the formation of cracks which parallel the pavement edge and progress laterally toward centreline (3).

Materials

The influence of materials on pavement deterioration can be looked at in two respects. The temperature susceptibility of asphalt cements is critical in areas where low temperatures are expected (4). While softer grade binders will reduce the probability of thermal cracking, stability may be sacrificed in terms of traffic associated distresses such as rutting.

The materials located within, and just below, the pavement structure, also have a significant bearing on pavement performance. Base courses which contain a high percent of fine particles such as silt and fine sand will be subjected to frost heaving during the cold season if moisture is present. This causes stresses to develop in the pavement and cracking will take place (4).

Design

The formation of distress features in a pavement is related to the type of pavement and supporting structure in question. In Ontario, as in many other jurisdictions, two basic pavement designs are used; flexible and rigid.

Flexible pavements range from a conventional single lift wearing course over base and subbase layers, to deep strength designs(150 mm or more of asphalt concrete) to full depth structures which could consist of up to 350 mm of compacted asphalt concrete placed directly on the subgrade.

Rigid pavements include concrete designs (concrete over base) and composite structures which consist of an asphaltic concrete wearing course over a concrete base over a lean concrete subbase. This latter design is a modification of the Ministry's original composite design which utilized cement treated Granular A as a subbase layer.

Each type of pavement is subjected to a particular set of distress features which in turn, have a bearing on the selection of rehabilitation alternatives. These distresses are described in detail in manuals produced by M.T.C. to assist its personnel in carrying out periodic condition evaluations of Ontario highways (5, 6).

REHABILITATION OF PAVEMENTS

As mentioned earlier, the level of serviceability of a new pavement begins to decline, almost from the day of construction. The rate of decline varies of course, depending on various factors, some of which have been discussed briefly. For each classification of highway, the designer must choose a level of performance, which when reached, triggers a need to carry out some form of rehabilitation, in order that serviceability may be restored to an acceptable level.

The principal method of upgrading the performance of a pavement is by placing a bituminous overlay. Reflection cracking, unfortunately is a major problem with respect to prolonging the life of an overlay. Past records indicate that, in Northern Ontario, cracks have reflected through single course resurfacing mats after one winter. If no attempts are made during the design and construction of overlays to control, or at least minimize, reflection cracking, highway authorities are faced with the following consequences:

* Premature drop in the level of serviceability
* Necessity for maintenance early in the life of the overlay
* Maximum benefits are not realized from the new asphalt concrete, an expensive portion of the total rehabilitative cost
* A waste of construction materials such as aggregates, is inevitable if the overlay life is shortened

It becomes extremely important, therefore, in a period when construction and maintenance of highways is subject to significant financial constraints, that measures be taken to prolong the life of asphalt overlays and protect the investment made in our roadway systems.

MECHANISMS FOR REFLECTION CRACKING

Flexible Pavements

In seasonal frost areas, one of the most serious types of distress which occurs in flexible pavements is thermally induced transverse cracking (4). This problem develops when the tensile strength of the bituminous material is exceeded by the stresses created as a result of rapid and extreme changes in temperature. In effect, the pavement cannot expand quickly enough to accomodate the temperature change. Observations have shown that cracks tend to close during the warm summer months and open as temperatures cool in the late fall and winter. This movement of the old pavement creates stresses in the underside of a new overlay (Figure 3). If the new mix is not flexible enough to accomodate this movement, a crack will form which, in time, may propagate through the lift to the surface. Once a crack develops, moisture can readily enter the base. Studies have shown that 70% of surface runoff can permeate cracks as little as 1.5 mm in width (1). This moisture softens the base which loses bearing capacity, causing the pavement to settle locally around the crack. Ejection of fine base particles under the dynamic loading of vehicles compounds the problem and accentuates this settlement or "dipping". During the cold season, moisture from melting snow and deicing operations accumulates in the cavity, freezes and forces the pavement to heave, a process referred to as "tenting". Once a reflection crack has opened up in an overlay, this combination of tenting and dipping contributes to weakening of the asphalt concrete and the formation of multiple cracks.

Rigid Pavements

When concrete pavements are constructed, contraction joints are formed or sawed in the new surface to minimize the formation of random transverse cracks during the curing period. In time, the concrete tends to crack through to the bottom of the slab at these locations. Intermediate cracks habitually form as a result of temperature fluctuations throughout the year. As cracks widen, the aggregate interlock at the faces of the slab is reduced and each segment of pavement becomes relatively free to move independently. As moisture accumulates beneath the pavement, the process of pumping takes place, whereby fine base and subgrade particles are both transported beneath the concrete, and ejected onto the surface of the slab. Figure 4 illustrates the development of stepping or "faulting" in concrete pavements. Experience has shown that heavy lifts of asphaltic concrete are essential to retard the formation of reflection cracks in these overlays.

TREATMENTS TO PREVENT REFLECTION CRACKING

Over the years, highway agencies have tried various treatments to prevent reflection cracking (4). Ontario has placed granular interlayers between the cracked pavement and new overlays. A limited use was made of wire mesh to reinforce a new asphalt concrete course placed over very severe longitudinal cracks, located on Hwy. 11, at Raith in northwestern Ontario. In northern Ontario, the introduction of softer asphalt cement grades met with some success in controlling reflection cracking.

Other jurisdictions have experimented with additives such as rubber and fibres, but with limited success.

TROUT CREEK EXPERIMENT

In a concerted effort to evaluate the potential of various treatments in controlling reflection cracking, the Ministry of Transportation and Communications constructed several test sections as part of a resurfacing contract near North Bay, in 1971. Eight trials were constructed as indicated in Figure 5.

Section 1, Screenings Interlayer

A 25 mm layer of crushed stone screenings (Table 1) was spread over the existing pavement prior to resurfacing. Distortions in the old surface created difficulty in controlling the depth of the screenings, and the paving equipment tended to disturb the stone. Ultimately, the lower binder course was increased from 40 mm to 50 mm, to give a total resurfacing depth of 112 mm.

Section 2, Granular A Interlayer

A 75 mm layer of Granular A (Table 1) was placed over the old pavement prior to paving with 100 mm hot mix.

Section 3, Granular A Interlayer

This section was similar to number 2, except that the granular layer was increased to 150 mm.

Section 4, Open Graded Binder Interlayer

The first hot mix binder course to be laid over the old pavement was made up of an open graded mix, using 100% crushed stone (Table 1).

Section 5, Existing Pavement Replaced

The cracked pavement was ripped up, pulverized through a crusher and reused as shouldering material. The base was restored with new material and paved with 100 mm of new asphaltic concrete.

Section 6, Existing Pavement Pulverized

In this section, the existing pavement was removed and crushed to meet the Granular A Specification as shown in Table 1. The processed asphalt was placed over the existing base material and compacted prior to resurfacing.

Section 7, Existing Pavement Pulverized and Enriched

The old pavement was pulverized as in Section 6. It was then stabilized with MC-250 cut back. The enriched material took some time to set up and repetitive windrowing and blading was required to speed up curing.

Section 8, Conventional Resurfacing

Section 8 was designated as a control section where the treatment consisted of normal resurfacing with 100 mm H.L. 4 hot mix.

Performance of Treatments

Crack surveys were carried out on the eight test sections on a yearly basis. Table 2 shows the density and severity of transverse cracking as recorded in July, 1976, five years after construction. Reflection cracking was most severe in Sections 1, 4 and 8 which contained screenings, open graded binder and conventional mix respectively. A lesser degree of cracking occurred over the Granular A interlayers while the most promising treatments involved the pulverization and relaying of the existing pavement as base material.

Conclusions

Having analysed the Trout Creek project from the standpoint of construction difficulties, performance data and long range economic implications, it was concluded that (i) pulverizing and relaying the existing pavement as base, and (ii) pulverizing, enriching and relaying the existing pavement as base, would be the most viable alternatives to conventional resurfacing of bituminous pavements.

RECENT DEVELOPMENTS IN THE CONTROL OF REFLECTION CRACKING

Since the Trout Creek project, the Ministry has investigated various other means to control reflection cracking. These efforts have been concentrated in three general areas:

- Geotextiles and Fabrics
- Modified Hot Mix Binder Layers
- Variable Overlay Depths

These applications have been tried over all three major pavement types, namely flexible, rigid and composite.

GEOTEXTILES - Flexible Pavements

The principle behind the use of a geotextile to control reflection cracking is reinforcement of the bituminous overlay. As discussed earlier in this paper, tensile stresses are induced in an overlay at points directly above cracks in the old pavement, which expand in cold weather. The geotextile, which is bonded to both old and new pavements, is intended to develop tensile strength under strain and acts like a buffer so that the movement or extension of the overlay is reduced from that experienced by the existing pavement.

Most reinforcing fabrics consist of a woven or non-woven mat, manufactured from polypropylene or polyester filaments. The Ministry first tested a geotextile for overlay reinforcement in 1977. Petromat, a non-woven polypropylene fabric was installed in two test sections on Hwy. 50, 50 km north of Toronto (7). The existing two lane pavement was severely distressed with multiple and braided longitudinal cracks and extensive alligatoring throughout (Figure 6). Petromat was placed in one lane for a distance of 90 m and in both lanes for 185 m. In both cases, a 25 mm H.L. 2 levelling course was placed on the existing pavement, followed by the geotextile and a 40 mm H.L. 4 wearing course (Figure 7).

Very little cracking occurred in, or adjacent to, the test sections during the first two or three years after installation. Table 3 summarises data from a crack survey taken in June, 1982.

A second installation of reinforcing fabric was made on Hwy. 629, outside of Timmins, in 1979. The product used was Bidim C-28, a non-woven polyester mat manufactured by Monsanto Textiles Company. The Bidim was placed between a 20 mm H.L. 2 levelling course and a 40 mm H.L. 4 surface course.

Table 4 contains data from a survey done in July of this year. It would appear that there is 23% more cracking in the section containing the reinforcement, in contrast to the control sections. Since the entire length of Hwy. 629 is still in good condition, more observation will be necessary before definite conclusions can be drawn as to the effectiveness of the geotextile at this site.

GEOTEXTILES - Composite Pavement

In 1979, Petromat was installed in one of several test sections located on a two lane highway near Winchester in Eastern Region. Constructed in 1968, the pavement structure included a 150 mm cement treated base and 160 mm asphalt concrete surface. Shrinkage cracks which developed in the base shortly after construction, had reflected through the wearing course and were contributing to a rough ride.

In view of the fact that 145 km of equivalent 2 lane highway, which had been constructed with cement treated base in Eastern Region since 1968, would soon be requiring some rehabilitation, it was decided to try several techniques and designs to retard or prevent reflection cracking in overlays (8).

The Petromat was placed between two 40 mm lifts of H.L. 4 hot mix. Asphalt cement was applied as a tack coat for the fabric.

After two years, observations revealed that almost all of the cracks in the old pavement had reflected through the overlay. Results will be discussed more fully later in the paper.

GEOTEXTILES/MEMBRANES - Concrete Pavements

The experimental use of Geotextiles by M.T.C. to control reflection cracking caused over concrete pavements, has involved a strip or "bandaid" type treatment, where the fabric has been applied solely at cracks and joints.

An initial trial was carried out on Hwy. 401, just west of Toronto, in 1975. A material called Bituthene H.D.G., consisting of a reinforcing polypropylene mesh laminated to 1.7 mm thick layer of self-adhesive rubberized asphalt, was applied to all joints and cracks in a 2 lane, 460 m test section. The normal depth of 125 mm of asphalt concrete was reduced to 75 mm in the test area to see if the use of Bituthlene would allow similar reductions in future work. (9).

A crack survey was completed a year later with results shown in Table 5. The membrane did not prevent reflection cracking from occurring; however, its performance was comparable to the conventional ovèrlay. Pavement cores drilled through several reflected cracks revealed that the Bituthene was still intact across the original crack, thus providing an effective barrier to surface moisture that might otherwise enter the base and contribute to further stepping.

Due to reconstruction, monitoring was discontinued shortly afterwards.

A second trial involving a membrane similar to Bituthene, was installed on Hwy. 401 near Morrisburg, in 1980. The material, known as PrePave, consisted of a 3.2 mm thick layer of reinforced asphalt sandwiched between two layers of asphalt coated polypropylene. A coating of 85/100 grade asphalt cement was used to bond the PrePave to the concrete. All joints and cracks in two 380 m test sections were covered with 500 mm strips of the material. Table 6 shows the depths of resurfacing in the test areas. These depths were somewhat less than the conventional 125 mm in order to see if the reinforcement could replace a portion of the overlay (Figure 8).

The results of surveys carried out since construction (Table 6) show that the PrePave did not prevent reflection cracking nor was the performance of either test section superior to the control area.

MODIFIED HOT MIX BINDER COURSES

Open graded asphalt concrete mixes, or "modified" hot mixes, have been used for several years as binder courses in the Toronto area. Specifically, these mixes have been placed as lower binder courses within the overlay when badly stepped concrete pavements were rehabilitated.

The large percentage of voids in the material helps to dampen the movement of the concrete slabs and hence reduces the stresses created in the upper binder and wearing courses. The open graded binder bridges open joints and intermediate cracks. Various gradation specifications for modified hot mixes used in southern Ontario are given in Table 7 . The asphalt content of these open graded mixes is substantially lower than in conventional asphalt concrete, and further savings are realized.

In 1980, the Ministry constructed its first trial section in which an open graded H.L. 4 mix was placed over a cracked bituminous pavement (10). The asphalt cement was set at 3%. Allowing sufficient time for cooling and setting up, the construction process was successful. Several more resurfacing contracts were completed in 1981, using a modified H.L. 4 in the binder layers.

VARIABLE DEPTH RESURFACING

In the preceding section, it was stated that modified mixes were first included in heavy, 125 mm overlays covering concrete pavements. In 1980, test sections were constructed on Hwy. 401 at Moulinette Road in which the total overlay depth was cut back to see if a thinner pavement, in combination with a modified binder or reinforcing membrane, could significantly affect the degree of reflection cracking. This particular project was referred to in earlier discussions on PrePave.

Table 6 outlines all the treatments and design modifications that were incorporated into the project, and results from two years of observations. There is an indication that the modified H.L. 8 mix has performed well, even when the total overlay depth was reduced. The least amount of cracking, however, has occurred in the control section.

WINCHESTER TEST SITE

Earlier in the paper, reference was made to a series of test sections constructed over a composite pavement near Winchester, Ontario. Table 8 summarizes the designs and performance data after three years of observation. It is obvious that none of the measures used will be very effective in preventing reflection cracking over a cement treated base.

MINIMIZING EFFECTS OF REFLECTION CRACKING

In the introduction to the paper, the adverse effects on pavement performance as a result of reflection cracking were dealt with in depth. The entry of moisture into the base and subbase layers, and subsequent loss of strength in the pavement structure, is, without doubt, one of the most serious consequences.

Realizing that some reflection cracking is inevitable in seasonal frost areas, the Ministry has taken steps to try and minimize any harmful effects that Ontario highways might suffer.

Crack Sealing

A greater emphasis is being placed on the sealing of cracks which reflect into new overlays. Routing and sealing is being actively carried out by maintenance staff,and via sealing contracts (figure 9). Many major resurfacing contracts now include sums of money designated for future crack sealing work. New sealing techniques and products are continually being evaluated.

During the past year, cracks were sealed in existing pavements prior to resurfacing on a few projects. The intent was to prevent moisture from entering the pavement structure until such time as any reflected cracks could be routed and sealed in an efficient manner.

Pavement Drainage

Asphaltic concrete overlays are subjected to high stresses when placed over badly faulted concrete pavements. Since excess moisture is a prime contributing factor to faulting, the Ministry of Transportation and Communications has been following a policy of installing perforated plastic subdrains along concrete pavements. This work has been incorporated into resurfacing projects as a rule, although a few contracts have been awarded specifically for edgedrain work.

The edgedrains are installed by ploughing (Figure 9), a technique first tried by the Ministry in 1977 (11). By the end of the current construction season, approximately 1954 km of plastic tubing will have been ploughed into Ontario highways. Priority has been given to draining existing concrete and composite pavements, however edgedrains are now a standard feature in all new composite and full depth asphalt pavements (Figure 10).

CONCLUSIONS

As new pavements age, the level of serviceability begins to drop until a point is reached when some form of rehabilitation is necessary. The most common type of rehabilitation is by applying an asphaltic concrete overlay. In a seasonal frost area such as Ontario, reflection cracking is a major concern with respect to achieving the desired level of performance throughout the design life of the overlay.

In the early seventies, experimental work showed that various types of granular interlayers and special bituminous mixes were not too successful in preventing thermal induced reflection cracking. The best performance was achieved when the old pavement was pulverized and reused as base.

Control of reflection cracking using reinforcement geotextiles has proved to be only marginally successful in the limited number of applications made by the Ministry on flexible pavements. More observation will be required before final conclusions are drawn.

A geotextile was unsuccessful in preventing reflection cracking over a composite pavement, and similar results would be expected where concrete pavements are involved.

Reinforcing membranes have provided only fair control of reflection cracking when applied to stepped concrete pavements. The main benefit of these fabrics may be derived from their ability to prevent moisture from entering the base by providing a waterproofing barrier across open joints and cracks.

The most effective technique employed by the Ministry to date in its attempt to control reflection cracking has been the use of modified hot mixes as lower binder courses.

This has proved to be successful in the resurfacing of concrete pavements and early indications are that modified mixes will be equally as effective in deterring reflection cracking in bituminous pavements.

Trials utilizing overlay depths less than the conventional deep strength design of 125 mm to 130 mm have demonstrated that, for effective control of cracking, the heavier resurfacing is required.

While continuing to investigate means of controlling reflection cracking, the Ministry has recognized that efforts must also be made to minimize adverse effects on the highway system due to this problem. Crack sealing is being carried out extensively to keep moisture out of the pavement structure, and edgedrains have proved to be beneficial in expediting drainage of the bases and subbases.

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TABLE 1 GRADATION SPECIFICATIONS FOR MATERIALS USED AT TROUT CREEK SITE

SIEVE	SCREENINGS % Pass.	GRANULAR A % Pass.	H.L. 4 % Pass.	H.L. 4 (Modif) % Pass.
22.4 mm		100		
16.0 mm		75 - 100	100	100
13.2 mm		65 - 90	82 - 95	-
9.5 mm	100	-	58 - 83	30 - 52
# 4	50 - 100	35 - 55	40 - 67	0 - 10
# 8	-	-	28 - 67	-
# 16	20 - 55	15 - 45	16 - 60	-
# 30	-	-	8 - 47	-
# 50	10 - 30	5 - 22	4 - 27	-
#100	-	-	1 - 10	-
#200	0 - 12	0 - 8	0 - 7	0 - 2

TABLE 2 DENSITY AND SEVERITY OF TRANSVERSE CRACKING ON VARIOUS TREATMENTS AT TROUT CREEK EXPERIMENT, JULY 1976

SECTION NO.	TRANSVERSE CRACKS PER MILE	SEVERITY - CRACKING VS. CRACK WIDTH		
		13.0 mm	13.0 - 6.4 mm	6.4 mm - HAIRLINE
1	211	5%	80%	15%
2	106	6%	70%	24%
3	76	2%	33%	65%
4	239	3%	71%	26%
5	31	6%	38%	56%
6	12	-	17%	83%
7	8	-	14%	86%
8	294	16%	52%	32%
ORIGINAL PAVEMENT	1000+	50%	40%	10%

TABLE 3 CRACK SURVEY ON PETROMAT TRIALS - HWY. 50

SECTION	TOTAL LENGTH OF CRACKING (m)	SEVERITY - % CRACKING VS. CRACK WIDTH		
		0 - 6.4 mm	6.5 - 13.0 mm	13.1 - 19.0 mm
NO. 1				
WITH PETROMAT	15	73	27	0
CONTROL	65	35	65	0
NO. 2				
WITH PETROMAT	12	100	0	0
CONTROL	41	29	71	0

TABLE 4 CRACK SURVEY ON HWY. 629 AT TIMMINS

SECTION (@ 50 m)	TOTAL LENGTH OF CRACKING	SEVERITY - % CRACKING VS. CRACK WIDTH			
		0-6.4 mm	6.4-13.0 mm	13.0-19.0 mm	19.0 mm +
CONTROL NO. 1	39	5	60	35	0
BIDIM TREATMENT	65	21	68	11	0
CONTROL NO. 2	43	0	53	39	8

TABLE 5 CRACK SURVEY ON HWY. 401 WEST OF TORONTO, PAVEMENT IS CONCRETE

SECTION	FULL TRANSVERSE CRACKS			HALF TRANSVERSE CRACKS			JOINTS		
	Apr/75	Aug/76	% Refl.	Apr/75	Aug/76	% Refl.	Apr/75	Aug/76	% Refl.
BITUTHENE MEMBRANE + 75 mm HOT MIX	39	15	39	58	9	16	13	4	31
CONTROL - 125 mm HOT MIX	32	7	22	44	9	21	10	3	30

TABLE 6 SUMMARY OF CRACK SURVEY ON HWY. 401 AT MOULINETTE ROAD

SECTION	CRACKS				JOINTS			
	Apr/80	Jun/81	Jun/82	% Refl.	Apr/80	Jun/81	Jun/82	% Refl.
Control - 40 mm H.L. 3 Mod. 40 mm H.L. 4 40 mm H.L. 8 Mod.	90	20	20	22	40	2	2	5
Test No. 1 - 40 mm H.L. 3 Mod. 50 mm H.L. 8 Mod. PrePave	102	22	24	24	43	12	12	28
Test No. 2 - 40 mm H.L. 3 Mod. 75 mm H.L. 8 Mod.	89	24	28	31	44	3	4	9
Test No. 3 - 40 mm H.L. 3 Mod. 40 mm H.L. 8 PrePave	63	36	46	73	59	19	20	34
Test No. 4 - 40 mm H.L. 3 Mod. 40 mm H.L. 8 Nicolon Fabric	23	8	8	35	18	10	12	67

TABLE 7 M.T.C. GRADATION SPECIFICATIONS FOR MODIFIED HOT MIX

M.T.C. SIEVE DESIGNATION	PERCENT PASSING BY MASS H.L. 4	H.L. 8
37.5 mm		100
26.5 mm		70-100
19.0 mm		50-100
16.0 mm	100	43-90
13.2 mm	70-86	-
9.5 mm	30-52	15-55
4.75 mm	5-15	5-15
2.36 mm	0-10	0-10
75 µm	0- 4	0- 4

The percent by mass of Asphalt Cement shall fall between 2.5 and 3.5.

TABLE 8 SUMMARY OF TREATMENTS AND PERFORMANCE ON HWY. 43

Treatment	Trans. Cracks May/70	Reflected Cracks			
		June/80	July/81	July/82	% Reflection
Section 1 -- 75 mm H.L. 4 100 mm Crushed (Type 'B')	33	2	23	25	76
Section 2 -- 75 mm H.L. 4 Crack Sealing	44	28	43	44	100
Section 3 -- 40 mm H.L. 4 Petromat 40 mm H.L. 4	44	18	40	44	100
Section 4 -- 75 mm H.L. 4 Pulverized Asphalt Concrete	39	28	38	39	100
Section 5 -- 40 mm H.L. 4 75 mm Stabilized Crushed (Type B)	45	25	40	43	96
Section 6 -- 75 mm H.L. 4 (Control)	43	37	43	43	100

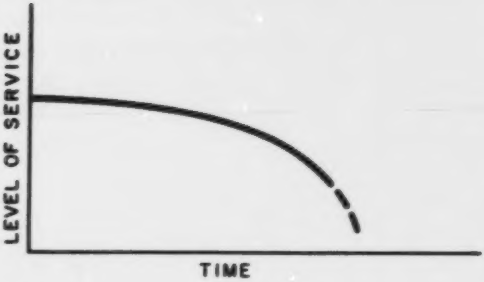


Figure 1/ Level of Service Drops as Pavement Ages

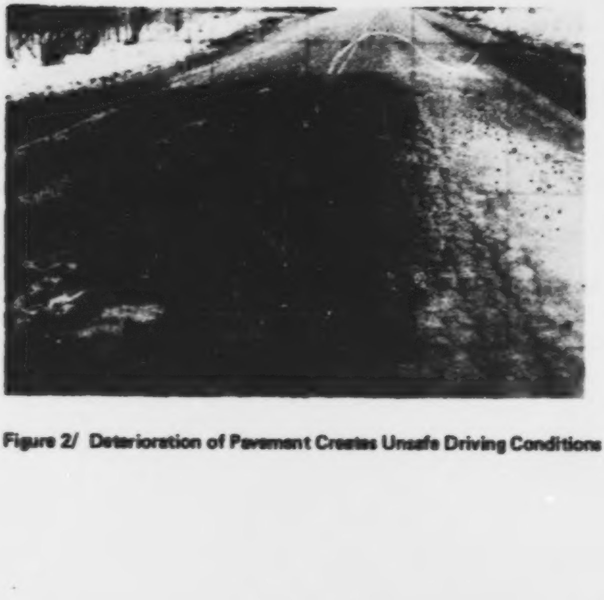


Figure 2/ Deterioration of Pavement Creates Unsafe Driving Conditions

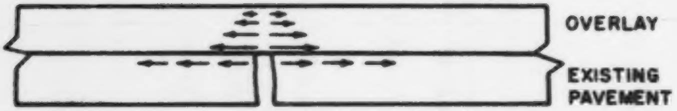
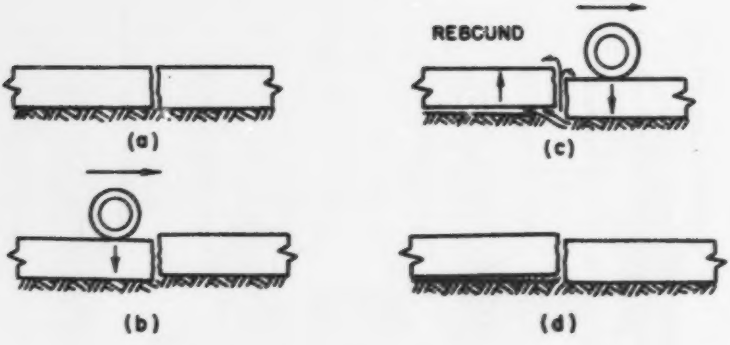

OVERLAY
EXISTING PAVEMENT

Figure 3/ Stresses Set Up in Overlay Due to Movement of Old Pavement


REBCUND
(a)
(b)
(c)
(d)

CONCRETE PAVEMENT FAULTING

Figure 4/ a) Loss of Aggregate Interlock at Open Joints and Cracks b) Leading Slab Depressed Under Load c) Movement of Fines Beneath Pavement d) Permanent Step at Crack

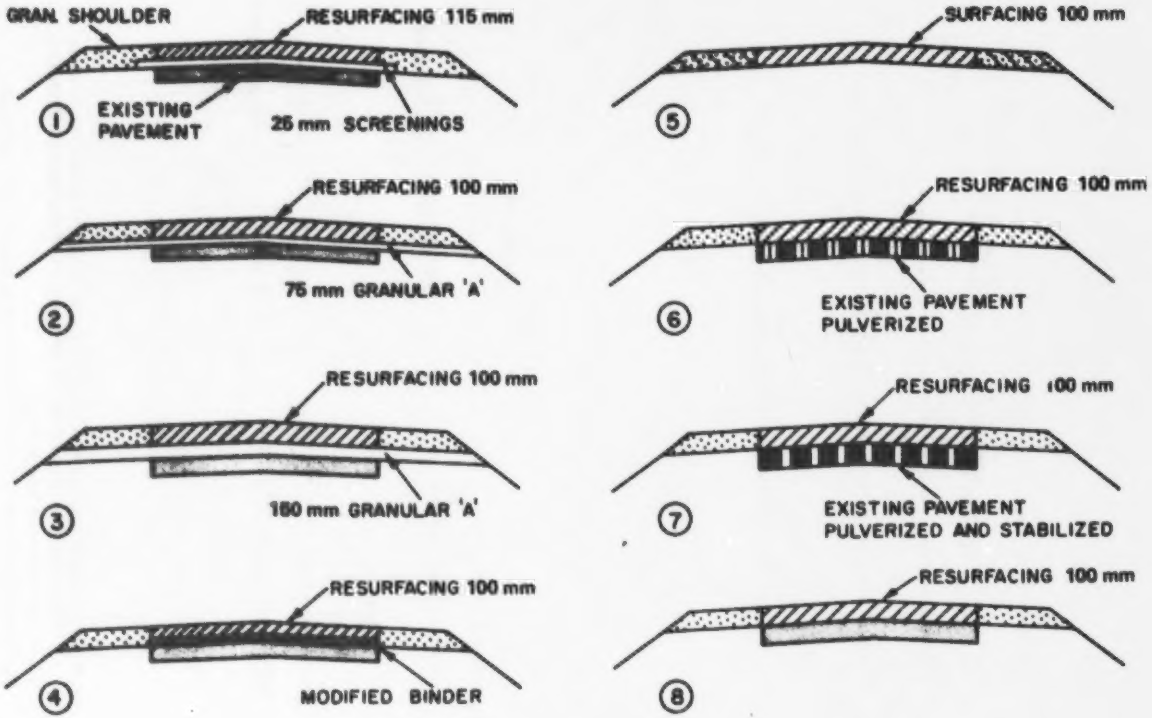

GRAN. SHOULDER
RESURFACING 115 mm
① EXISTING PAVEMENT
25 mm SCREENINGS
RESURFACING 100 mm
75 mm GRANULAR 'A'
②
RESURFACING 100 mm
160 mm GRANULAR 'A'
③
RESURFACING 100 mm
MODIFIED BINDER
④
SURFACING 100 mm
⑤
RESURFACING 100 mm
EXISTING PAVEMENT PULVERIZED
⑥
RESURFACING 100 mm
EXISTING PAVEMENT PULVERIZED AND STABILIZED
⑦
RESURFACING 100 mm
⑧

Figure 5/ Treatments Used in Trout Creek Experiment in 1971

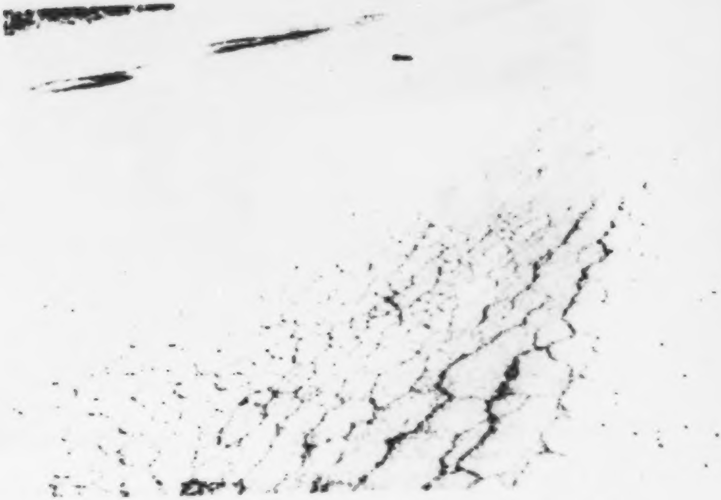


Figure 6/ Extensive Alligatoring on Hwy. 50 at Petromat Test Sites

Figure 7/ Placing Petromat on Hwy. 50 Test Site

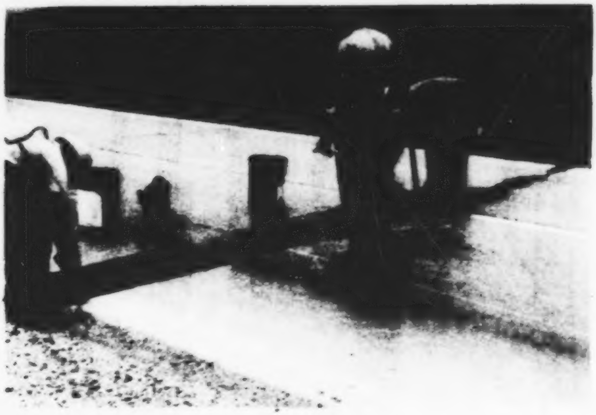


Figure 8/ Placing PrePave Membrane Over Joints and Cracks in Concrete Pavement

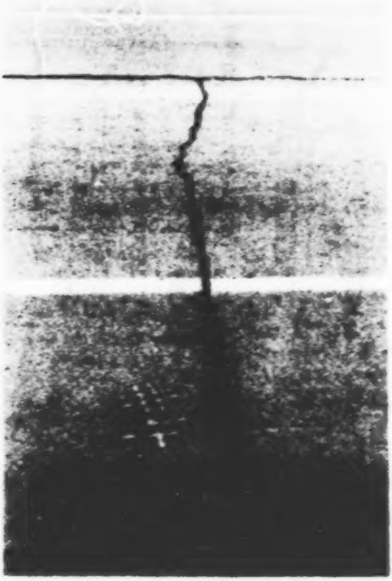


Figure 9/ Sealing Cracks with Hot-Poured Sealant

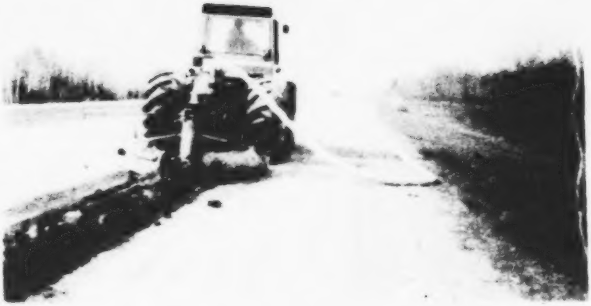


Figure 10/ Installing Edgedrains Using Ploughing Technique

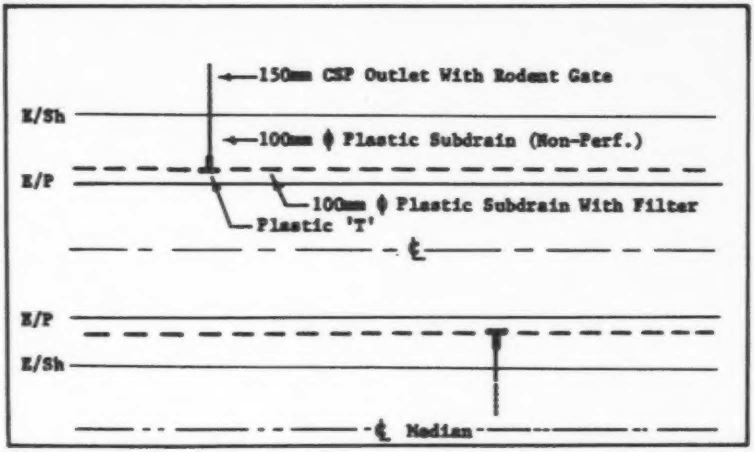

150mm CSP Outlet With Rodent Gate
E/Sh
100mm ϕ Plastic Subdrain (Non-Perf.)
E/P
100mm ϕ Plastic Subdrain With Filter
Plastic 'T'
℄
E/P
E/Sh
℄ Median

Figure 11/ Layout of Typical Edgedrain System in Ontario

Session 4

Paving Maintenance, Rehabilitation, and Reflection Crack Prevention Methods

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ABSTRACT

The present state of techniques used to prevent reflection cracking in Japan is described and the use of fabric sheets as a preventive technique is evaluated in this report.

1/ The present state of preventive techniques for reflection cracking.

The design and construction procedures of asphalt concrete overlay for existing asphalt and cement concrete pavements are described in the "Road Maintenance Manual," the outline of which is as follows.

- Overlay on existing asphalt concrete pavements.
  Two thickness design methods, the subgrade CBR method and the Benkelman deflection method, are used in Japan. Prior to overlaying, cracked areas must be treated by patching or planing. Planing is most widely used on heavy traffic roads.

- Overlay on existing cement concrete pavements.
  The overlay thickness is designed using subgrade CBR values, in which 8 cm is recommended for the minimal overlay thickness. To prevent occurrence of reflection cracking, open-graded asphalt concrete is used for binder course, or some kinds of fabric sheets are placed underneath the overlay.

2/ Evaluation of the preventive technique for reflection cracking with fabric sheets.

Two types of fabric sheets are used for this purpose: one is bituminous waterproofing sheets, and the other is fabric sheets placed directly on the pavement surface with bituminous adhesives. Both of them have proved to be effective in delaying the appearance of reflection cracks; however, they, so far, have failed in complete prevention. More reliable types of sheets are still in the development stage.

PAVING MAINTENANCE, REHABILATION, AND REFLECTION CRACK PREVENTION METHODS

by

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Public Works Research Institute, Ministry of Construction
Saburo Matsuno
Kanazawa University

1/ PREFACE

As a result of road construction efforts from the 60s into the 70s, the percentage of paved principal roadways increased rapidly. Table 1 shows the extension of the kinds of pavement in Japan, from which it can be seen that road extensions amount to 1,100,000 km and that pavement extensions account for about 43%. However, this 43% includes all of the so-called high-standard pavements based on the Asphalt Pavement manual and Concrete Pavement manual with other pavements based on the Low Cost Pavement manual also included. Thus, if low cost pavements are excluded, the percentage of paved roads becomes only 16%.

The length distribution of the paved roads by traffic, excluding expressways, is shown in Table 2, in which the percentage of national highways is about 40% for Traffic B (heavy vehicle traffic volume of 250 to 1,000 vehicles/day/direction), 24% for Traffic C (heavy vehicle traffic volume of 1,000 to 3,000 vehicles/day/direction) and 20% for Traffic A (100 to 250 vehicles/day/direction). On the other hand, most of prefectural and municipal roads have the Traffic L, and the percentage of sections with the traffic volume greater than Traffic C is only several percent.

However, as a result of the increased traffic volume of heavy vehicles in recent years, sections of B, C and D tend to increase only gradually.

Accordingly, the ratio of maintenance and rehabiliation for each road type has increased, and studies on the proper frequency of maintenance, rehabiliation work and work methods are currently being carried out.

The paper will first describe the situations of maintenance and rehabilitation of the major national roads, and then report the outline of a design method for the overlay, and examples of the reflection crack prevention method using bituminous fabric sheets.

2/ MAINTENANCE AND REHABILITATION

2.1/ Survey Outline

Out of a total of 15,000 km of national highways controlled by the Ministry of Construction, about 10% of the total of the asphalt pavement sections was sampled for survey after consideration of the traffic conditions and the number of years in service. The methods, areas, costs and materials of the maintenance and rehabilitation work performed for one year in each of fiscal years 1978, 1979 and 19890 were surveyed for each section.

Table 3 shows the extensions and areas surveyed for each district and each traffic class. The results were obtained after a continuous survey for three years at the same places. The deviation of results due to locality and traffic class is considered to be very small, so these results can be considered to directly reflect average situations for the national roads.

2.1.1/ Maintenance Work

2.1.1.1/ Areas

Maintenance work means minor repair work of relatively small scale carried out as an emergency measure, and methods of work used include planing, surface treatment with a less than 3 cm asphalt concrete layer (this will be abbreviated as "asphalt concrete" hereinafter), and local reconstruction.

Percentages of maintenance work execution area expressed by Equation 1 were found to be 4.3% for ordinary areas, 5.9% for snowy and cold areas, and 5.0% for the whole section.

$$\text{Percentage of maintenance work execution areas} = \frac{\text{Total of maintenance work execution areas}}{\text{Total of surveyed areas}} \times 100(\%) \;\ldots\ldots\ldots\ldots(1)$$

Figure 1 shows the sums of the percentages of maintenance work execution areas calculated for every one year of road in service. The sum of the percentages for 10 years in the sections with traffic volumes smaller than Traffic B was about 33% for ordinary areas, about 29% for snowy and cold areas, and the total sum for more than 15 years was about 52% and 58%, respectively. Thus, differences due to locality are small. However, in the case of sections having Traffic C, the sums for 10 years were about 20% and 48%, respectively, and the total sum was about 51% and 94%, respectively, so that the difference due to locality is very large. In the same manner, the sum for 10 years in sections having Traffic D was found to be about 41% and 73%, respectively, with a considerable difference between them.

With respect to the whole section, it is considered that maintenance work will be performed for about 32% of the total area over 10 years and for about 64% for the whole period in the ordinary district, while maintenance work will be performed for about 57% of the total area over 10 years and about 95% for the whole period in snowy and cold districts. From this, it can be deduced that the frequency or the scale of maintenance work in snowy and cold districts is larger than that of ordinary districts.

Figure 2 shows the proportions of executed areas by work method. In ordinary districts, the proportion of the planing method increases considerably as the traffic becomes heavier, and the proportion of the surface treatment method with planing also increases. In the whole section, about one-quarter of the total area is done by the planing method, about one-third by the surface treatment method with planing, about one-third by the surface treatment method without planing, and the remainder is done by the local reconstruction method.

On the other hand, in snowy and cold districts, the proportion of the surface treatments without planing is extremely high, and the total of the proportions of both the planing and surface treatment with planing is about one-half that of in the ordinary district. In the Traffic D section, the proportion of the surface treatment methods with planing is about 80%, the maintenance work executed area in the section of Traffic D is $102 \times 10^3 m^2$, and the area proportion of the snowy and cold districts to the total repair area is not very large. Thus, for the whole section, about 30% is done by surface treatment methods with planing, about 60% is done by the surface treatment method without planing, and the remainder is done by planing and the local reconstruction method.

Also, in any district, the proportion of local reconstruction methods was about 3%.

2.1.1.2/ Materials

Figure 3 shows the proportions of areas by repair material used for the surface treatment work.

In ordinary districts, the proportion of dense-graded asphalt concrete with a maximum grain size of 13 mm is large and about 84% in the section having a traffic volume lower than Traffic B. In the case of sections having Traffic C and Traffic D, the proportion of dense-graded asphalt concrete with maximum grain size of 13 mm is large and is the same as in the case of the section having traffic volume lower than Traffic B. However, the proportions of coarse-graded asphalt concretes as a measure against fluidity, dense-graded asphalt concrete with maximum grain size of 20 mm, and gap-graded, dense asphalt concrete as a measure against plastic flow increased to about 2%, 11% and 19%, respectively, in the entire section of the ordinary districts.

Also, with respect to modified asphalts, the proportion of the area where asphalt was used with rubber was about 1.7% in the ordinary district and about 2.9% in the snowy and cold district. The proportion of the area where the asphalt was used with resin was about 0.1% in the ordinary district and zero in the snowy and cold district. These kinds of modified asphalt were used for only about 2.4% of the total area in the whole district.

2.1.1.3/ Costs

Figure 4 shows the proportions of maintenance costs by the work method expressed by Equation 2. However, the costs in this paper are all expressed with direct construction costs excluding the indirect costs.

$$\text{Maintenance cost} = \frac{\text{Total of maintenance cost}}{\text{Total of surveyed area}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

When comparing Figure 2, the proportions of the planing method and surface treatment method without planing decreased, and the proportions of the surface treatment method with planing and local reconstruction methods increased. The area done by the planing method is about 14% for the whole section but the cost for this area was about 7% of the total. In the same manner, area and cost proportions were about 51% and 43%, respectively, for the surface treatment method without planing; about 33% and 42%, respectively, for the surface treatment method with planing; and about 3% and 8%, respectively, for the local reconstruction method.

Also, maintenance costs increase as the traffic becomes heavier, and is 45 yen/m^2 (18 ¢/m^2) in the section with traffic lower than Traffic B, 63 yen/m^2(25.2 ¢/m^2) in the section with Traffic C and 71 yen/m^2 (28.4 ¢/m^2) in the section with Traffic D without considering the districts. However, in the ordinary district, the maintenance cost for the section having Traffic C is smaller than that for sections having traffic lower than Traffic B.

As for the maintenance cost by district, the costs for the snowy and cold districts are larger than those for the ordinary district except for the sections with traffic lower than Traffic B, and the cost for the whole section is about 1.25 times that of the ordinary district. Though the difference of 1.37 times (=5.9/4.3) is seen in the percentages of maintenance work executed areas, the difference of materials between both the districts is considered to be small. As to the methods of work, the proportion of the surface treatment method with planing in the snowy and cold district was smaller than that in the ordinary district, and the proportion of the surface treatment without planing in the snowy and cold district was larger than that in the ordinary district. Thus, the difference as stated above is considered to have occurred.

Planing was performed at 261 places, and its mean depth was distributed from 0.1 to 11.5 cm, but the mean depth of planing at 231 places was greater than 1 cm and less than 4.5 cm. And the mean of the mean planing depths was about 3.0 cm with no difference due to locality and traffic class. The unit planing price per unit depth was calculated by the method of least squares from the planing costs for 129 places and found to be 212 yen/m^2/cm (84.8 ¢/m^2) for the ordinary district, 149 yen/m^2/cm (59.6 ¢/m^2) for the snowy and cold district, and 186 yen/m^2/cm (74.4 ¢/m^2) for the whole district. These are the mean amounts for three years from 1978 to 1980 as described before.

From the above, the standard maintenance work is considered to be as indicated in Table 4. For example, if an area of 100 m^2 is assumed as a pavement section, it is considered that maintenance work is performed for 4.3% of it or for an area of 4.3 m^2 for one year in the ordinary district. And with this area of 4.3 m^2, planing work is performed for 1.0 m^2, surface treatment with planing for 1.5 m^2, surface treatment without planing for 1.6 m^2 and local reconstruction for 0.2 m^2, and the maintenance cost for each is 600 yen ($2.40(=2.4¢/m^{2}x100m^2)) (=6 yen/m^{2}x100m^2), 2,400 yen ($9.60), 1,800 yen ($7.20) and 600 yen ($2.40), respectively, with a total cost of 5,400 yen ($21.60).

Figure 5 shows the sums of maintenance costs for each year in service. The sum of maintenance costs for five years is 242 yen/m^2 (96.8 ¢/m^2), 555 yen/m^2 ($2.22/m^2) for 10 years, and 958 yen/m^2 ($3.83/m^2) for 15 years. The maintenance cost per year slightly increases as the number of years in service increases.

The snowy and cold districts tend to require a higher maintenance cost in relatively earlier stages compared to the ordinary district. In order to find the difference in time, maintenance costs for 10 years, based on an interest rate of 7%, were calculated using Equation 3:

$$S = (1.07)^9 C_1 + (1.07)^8 C_2 + \ldots + (1.07)^0 C_{10} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (3)$$

where, S = Maintenance cost for 10 years (yen/m^2) (¢/m^2)
C_i = Maintenance cost for pavement with number of years in service less than $(i-1)$ to i. (yen/m^2) (¢/m^2)

As a result, the maintenance cost in the snowy and cold district for 10 years is about 1.50 times (=683/456) that of the ordinary district without considering interest and is about 1.69 times (=952/564) with interest considered.

2.1.2/ Rehabilitation Work

2.1.2.1/ Areas

Rehabilitation work means the relatively large-scale repair work performed on the pavement which has reached a predetermined period of service, and this work is represented by the overlay. Where overlay is not appropriate, the reconstruction method is used.

Figure 6 shows the proportions of area by planing, overlay and reconstruction for districts and various traffic classes.

In the ordinary district, as the traffic becomes heavier, the proportion of area by planing increases and the proportion of area by reconstruction decreases. And, in the whole section, the proportions of 1-layer and 2-layer overlay methods are about 32% and 35%, respectively, which are almoct the same. About one-half of the area of each method is considered to be overlayed after planing.

In the snowy and cold districts, the proportion of the 2-layer overlay is larger than that of the 1-layer overlay, and the proportions of 2-layer and 1-layer overlay methods are about 66% and 16%, respectively, in the whole section and the ratio between them is 4:1. The proportion of the planing method is about 15% and is about one-half of that in the ordinary district.

The proportions of total areas by the overlay and reconstruction methods in the surveyed are are 8.0% for the ordinary district, 7.1% for the snowy and cold district, and 7.6% for the whole district.

2.1.2.2/ Materials

The proportions of areas by material used for the overlay work are shown in Figure 7.

In the ordinary district, coarse- and gap-graded asphalt concrete, which was used for about 19% of the area in the maintenance work, was used for only a small area in the case of overlay. In the case of 1-layer overlay work, the dense-graded asphalt concrete having maximum particle sizes of 13 mm and 20 mm was the most frequently used and amounted to about 68%. Also, in the section with Traffic D, the coarse-graded asphalt concrete was all rubberized but, in addition, the thermoplastic modified asphalt was used. Their proportions to the whole section were about 26% and 1.6%, respectively. In the case of 2-layer overlays, coarse-graded asphalt concrete was used for the base course in most cases and, ^not only dense-graded asphalt concrete, but also fine-graded asphalt and fine- and gap-graded asphalt concretes were used for the surface course. Modified asphalt was used for sections with Trafic C and Traffic D, and the proportion to the whole section was about 1.6% in the case of rubberized asphalt and about 0.9% in the case of thermoplastic modified asphalt.

In snowy and cold districts, a tendency similar to that of maintenance work was recognized. In the case of 1-layer work, dense-graded asphalt and fine- and gap-graded asphalt concretes were used but no percentages can be ascertained because of insufficient data. No modified asphalt was used. In the case of 2-layer work, coarse-graded asphalt concrete was used for most of the base course as for the ordinary district. However, for the surface course, dense-graded asphalt, fine-graded asphalt and fine- and gap-graded asphalt concretes were used for about 54% of total area. About 9% of this area was worked with rubberized asphalt.

2.1.2.3/ Costs

Mean depth of planing, overlay thickness and mean value of the number of years in service for the overlay work are shown in Table 6. The mean depth of planing tends to increase as the overlay thickness becomes larger, and was 2 to 5 cm in the case of 1-layer work, and 5 to 10 cm for ordinary districts and 3 to 5 cm for snowy and cold districts in the case of 2-layer work. Thus, there was a difference between the 1-layer and 2-layer in the ordinary district, and the mean value was about 5.5 cm which was about twice that of the maintenance work. The difference between the 1-layer and 2-layer was almost nil and the mean value was about 3 cm in the snowy and cold district, almost the same as that of the maintenance work.

The overlay thickness was about 5 cm for 1-layer and remains the same regardless of locality, but for 2-layer thicknesses was about 10 cm in the ordinary districts and about 8 cm in snowy and cold districts.

The number of years in service is the number of years between the time of the installation, the time of planing or the time of execution of previous overlay and the time of execution of the present overlay. The number of years in service in the case of 2-layer overlays tends to become larger than that of 1-layer, but the values of snowy and cold districts are about one year longer than that of the ordinary district. The number of years in service in the case of overlays with planing was shorter than that having overlay only. The number of years in service until the execution of reconstruction was about 11.1 years, on average, taking 46 places in the ordinary district, and about 11.3 years, on average, taking 28 places in snowy and cold districts.

Also, the unit overlay price per unit thickness was calculated by using the method of least squares from costs for places where only the overlay had been performed. The unit price was 269 yen/m^2/cm (\$1.07/m^2) for the ordinary district, 264 yen/m^2/cm (\$105.60/m^2/cm for the snowy and cold district and 267 yen/m^2/cm (\$106.80/m^2/cm) for the whole district.

From the above, in the ordinary district, the proportion of 2-layer overlay work was smaller, but the proportion of planing, mean depth of planing and overlay thickness were larger and the number of years in service until the execution of overlay was shorter than those of snowy and cold districts. Thus, it is considered that the cost of rehabilitation work for ordinary districts are higher than those for the snowy and cold districts. Thus, the standard rehabilitation work for each district was set as shown in Table 7 and then the rehabilitation cost was calculated. For example, the percentage of areas requiring rehabilitation work for snowy and cold districts was 7.1%, and the ratio in areas between the 1-layer and 2-layer overlay methods was 1:4 from Figure 6. Thus, 1.4% and 5.7% were obtained. Also, 1.4% was used because the area of the planing method was one-fifth that of the overlay work. The number of years in service and thickness were determined from Table 6.

Now, if a paved area of 100 m^2, unit overlay price of 270 yen/m^2/cm (\$1.08/m^2/cm) and unit planing price of 190 yen/m^2/cm (76 ¢/m^2/cm) is assumed, then the rehabilitation cost for each district can be calculated as shown below.

Ordinary District

Overlay (1-layer)
100m^2 x 4.0% x 270 yen/m^2/cm (\$1.08/m^2/cm) x 5cm = 5,400 (\$21.60)
Overlay (2-layer)
100m^2 x 4.0% x 270 yen/m^2/cm (\$1.08/m^2/cm) x 10cm = 10,800 (\$43.20)
Planing
100m^2 x 4.0% x 190 yen/m^2/cm (76 ¢/m^2/cm) x 5.5cm = 4,180 (\$16.72)

Total 20,380 yen (\$81.52)

Snowy and Cold District

Overlay (1-layer)
100m^2 x 1.4% x 270 yen/m^2/cm (\$1.08/m^2/cm) x 5cm = 1,890 (\$7.56)
Overlay (2-layer)
100m^2 x 5.7% x 270 yen/m^2/cm (\$1.08/m^2/cm) x 8cm = 12,312 (\$49.24)
Planing
100m^2 x 1.4% x 190 yen/m^2/cm (76 ¢/m2/cm) x 3cm = 978 (\$3.91)

Total 15,000 yen (\$60)

Thus, the rehabilitation cost per year is 1,940 yen ($7.76 (= $81.52 ÷ 10.5)) (= 20,380/10.5) for the ordinary district and 1,300 yen ($5.20($60/11.5)) (15,000/11.5) for snowy and cold districts. And the rehabilitation cost for the ordinary district is considered to be about 1.49 times that of the snowy and cold district. This is probably caused, as shown in Table 3, because the overlay thickness, mean depth of planing and area of planing become larger since the proportion of heavy traffic sections and the proportion of urban sections of the ordinary district are larger than those for snowy and cold districts.

3/ OUTLINE OF MAINTENANCE AND REHABILITATION METHODS

The situation of maintenance and rehabilitation was described in Section 2, but the surface treatment mostly used for maintenance and overlay work is also mostly used for rehabilitation work. For the design and construction of the methods, the Road Maintenance Manual (revised and issued in 1978) was used as a reference [1].

The results of surveys and experience were collected and compiled when preparing this Manual. The design method mainly for the execution of overlay will be outlined below, in particular for asphalt and Portland cement concrete pavements.

3.1/ Overlay on Asphalt Pavement

3.1.1/ Evaluation of Pavement and Selection of Maintenance Method

For the evaluation of pavement, its maintenance and rehabilitation, a PSI given by the following formula is used:

$$PSI = 4.53 - 0.518 \log\sigma - 0.371\sqrt{C} - 0.174D^2 \quad \dots\dots\dots\dots\dots\dots\dots (4)$$

where, PSI = Present Serviceability Index

σ = Standard deviation of longitudinal roughness (mm)

C = Percent of cracked area (%)

D = Mean depth of rutting (cm)

Figure 8 shows the relationship between the PSI and the corresponding maintenance method. According to the results of studies in recent years, a newer index, MCI, was proposed. Table 9 shows the decision criteria for the maintenance and rehabilitation of each type of failure. Figure 8 shows several types of maintenance and rehabilitation methods based on the percentage of cracked area traffic volumes. In selecting a particular method of maintenance and rehabilitation, an overall judgement is necessary by taking account of the site conditions, budget and by using Tables 8 and 9 and Figure 8.

3.1.2/ Design and Construction Method for Overlay

In the design of overlay thicknesses, two methods are normally used:
(1) method by CBR for subgrade, and
(2) method by Benkelman beam deflection.

The method by CBR for subgrade will evaluate the failed pavement by using the coefficients of relative strength of Table 10 to determine the value of T_{AO}, and use the difference t from T_A[2] required for the newly-constructed pavement as overlay thickness. That is,

$$t = T_A - T_{AO} \quad \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots \quad (5)$$

where, t = Overlay thickness (cm)
TA = Equivalent thickness (cm) for subgrade CBR
TAO = Equivalent thickness for failed pavement (cm)

In the case of the Benkelman beam deflection method, the deflections (5 tons of wheel load) of failed pavement are measured at 10 m intervals and data is arranged by the formula shown below.

$$D = (\bar{d} + 2\sqrt{V}) \times f \quad \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots \quad (6)$$

where, D = Deflection of failed pavement (mm)
$\bar{d}$ = Average of deflections measured (mm)
$\sqrt{V}$ = Standard deviation of measured values (mm)

f = Correction factor by the mean temperature of asphalt concrete layer

Table 11 indicates the relationship between the deflection D and the thickness of overlay. Two methods described above will be applied to the roads with heavy traffic and the thickness of overlay will be finally determined from an overall point of view.

In the asphalt pavement on roads with heavy traffic in this country, failures such as rutting due to permanent deformation of the surface and binder courses and cracks beginning from the top of the surface course occur frequently. Thus, the overlay maintenance method which planes the failed surface course or binder course is more frequently used. The design of the thicknesses of overlays for this method is performed in the same manner as that for the ordinary overlay already described. The surface recycling method has also been tried in recent years.

In the ordinary overlay method, considerably-failed portions are first treated by patching, levelling or reconstruction and then overlay is performed.

3.2/ Overlay on Portland Cement Concrete Pavement

3.2.1/ Evaluation of Pavement and Selection of Maintenance Method

Table 12 shows the decision criteria for the maintenance and rehabilitation of each type of failure. Also, Figure 9 shows the criteria for the selection of the maintenance and rehabilitation method. In the actual selection of the maintenance method, an overall judgement shall be made using Table 12 and Figure 9 taking into account the site conditions and budget. Generally, overlay is performed at an early stage and a thorough reconstruction is only occasionally performed. In the case of overlays, slabs considerably failed are reconstructed or, in many cases, asphalt or cement is grouted underneath the slabs.

3.2.2/ Design and Construction of Overlay

In the design of the thickness of the overlay, T_{AO} is determined using the coefficients of relative strength shown in Table 10 and the thickness is designed by the same method used for the asphalt pavement on the basis of subgrade CBR. Therefore, an overlay with a thickness of 5 to 10 cm is frequently designed except in cases where the subgrade is extremely weak. Where the traffic volume is small, an overlay 3 to 5 cm thick is first used because of its economy and then additional overlays are made sequentially depending upon the conditions of reflection cracks.

In order to prevent reflection cracks, it is recommended to use an open-graded asphalt concrete for the binder course or to place bituminous fabric sheets to the joints or cracked portions of slabs. The bituminous fabric sheets are usually waterproofing fabric sheets made of cotton or polypropylene cloth coated with blown asphalt. Prior to the placement of the sheets, dirt or mud is completely removed from the joints and cracks, and then the joints and cracks are filled with a mixture of sand and asphalt emulsion. Sheets are placed together with the asphalt emulsion and then the road is opened to traffic for more than one day to tightly bond the sheets to the slabs.

4/ EXAMPLES OF THE REFLECTION CRACK PREVENTIVE METHOD

4.1/ Reflection Crack Preventive Method Using Bituminous Fabric Sheets

Figure 10 shows the case of cracks occurring on the overlays of concrete pavement on national highways [3]. The situation of concrete pavement and overlay is shown in Table 13. If the traffic volume of heavy vehicles after overlay is 5 million, for example, Figure 10 shows that about 30% of the surveyed places has more than 5% cracked area. In addition, about 20% of the surveyed places has more than 10% cracked area. If sheets are used, about 20% of the surveyed places has more than 5% cracked area in cases where heavy vehicle traffic volume is 5 million. This means that the sheets are effective for preventing the reflection cracks in the early stages of overlay. However, from a different point of view, it is difficult to fully prevent reflection cracks.

4.2/ Reflection Crack Preventive Method with Bituminous Fabric Sheets Using Cut-Back Blown Asphalt

The asphalt used for bituminous fabric sheets is made of blown asphalt having a penetration of 10 to 20, which has been cut back by lubricating oil and turned into mastic by adding slacked lime, and its properties are shown in Table 14. Then, this asphalt is coated to a piece of hemp cloth to form a 3-mm thick bituminous fabric sheet. Some sheets have the chippings 5 to 10 mm in size bonded under pressure to their surfaces. Features of these sheets include the cut-back blown asphalt and its larger thickness than that of conventional bituminous waterproofing fabric sheets.

When applying the above sheets to the concrete pavement, the joints and cracked portions are filled with cut-back blown asphalt the same as that used for the sheets, and then the sheets are cut into widths of 30 cm and placed with the same asphalt. The road is opened to traffic for one to two days to tightly bond the sheets before the overlay. Since the sheets are placed with hard asphalt, they neither move nor are peeled off by the traffic. Photo 1 shows the situation of sheets placed on the joints of concrete pavement on which faulting had occurred and then opened to traffic for one year. No cracks are seen in Photo 1 but hair cracks are seen in Photo 2. These sheets are considered to have been subjected to more severe conditions than those of other sheets placed below an overlay. However, in order to fully prevent reflection cracks, the sheets should not cause any cracking to occur even if they are directly subjected to traffic.

Table 15 shows the occurrence of reflection cracks in overlays in which the above sheets were used. According to the survey conducted in 1982, the occurrence of reflection cracks is relatively low. Section No. 2 overlay of thickness 3 cm was made by planing and overlay as an approach to the freeway. Seven years later, this section had only insignificant reflection cracks before the reconstruction in 1982, as can be seen in Photo 3.

When using sheets over an asphalt pavement, the cracked portion should be sealed, have 1-m wide sheets placed, and be opened to traffic for one to two days, and then an overlay should be made as in the case of concrete pavement. Photos 4 and 5 show a situation where the sheets were placed onto the cracked portion of asphalt pavement and then opened to traffic. In each case, pot-holes occurred and patching was done. This problem is likely caused by the stripping of the asphalt concrete mixture beneath the sheets. Therefore, unlike concrete pavement, the provision of a ventilator to allow the easy escape of vapors from the cracked portion seems to be required.

Photos 6 through 10 show a thin overlay (1.5-cm thick) made after placing the sheets onto the cracked portion. Slight reflection cracks occurred in this case seven years later. Photo 11 shows a portion of pavement one year later in which no sheets were used and in which reflection cracks fully developed. In view of the heavy traffic, it is considered that the sheets were very effective.

The number of examples of applications of bituminous fabric sheets using cut-back blown asphalt is still few so it is difficult to draw a perfect conclusion at present. However, a few suggestions for the prevention of the reflection cracks are summarized below.

1/ Asphalt used for the bituminous fabric sheets should result in a minimum of cracks. Thus, it is better to give some kind of treatment instead of using blown asphalt as it is. In particular, a possible use of mastic-type asphalt should be considered. When comparing these sheets to each other, it is better to make a comparison by subjecting them to direct traffic as shown in Photos 1 and 2.

2/ The thickness of sheets seems to be related to the records of their service. It is desirable to give a certain minimum thickness and stiffness to the sheets but this will vary depending upon the bituminous materials used. Also, a thickness that may cause considerable vibration to occur when opened to traffic is not desirable. A thickness of 3 mm was able to reduce the vibration, and good drivability was maintained.

3/ Joints and cracked portions must be filled with asphalt mortar or hard asphalt, and they should not be depressed after opening to traffic. They should be opened to traffic for several days after the filling, and sealing should be given again if the filling is insufficient.

4/ If the sheets are placed with the asphalt emulsion, in many cases, sheets move after opening to traffic. Thus, it is more desirable to place the sheets using hard asphalt. However, in order to relieve the stress concentration to the overlay, further study of the properties of asphalt is required.

5/ If the sheets are placed over the asphalt pavement, stripping of the asphalt concrete mixture may be hastened due to moisture in the cracked portion. It is not clear how and in what degree the service performance of the overlay is affected by this stripping. However, it seems necessary to provide some kind of ventilating means.

6/ In the case of applications to the top of asphalt pavement, the cost of the bituminous fabric sheet maintenance method is lower than the cost for reconstruction after digging up the cracked portion and is slightly higher than the cost of giving surface treatment with a mixture directly over the cracked portion. Therefore, the comparison of the service performances between the surface treatment method and bituminous fabric sheet method will become an important factor when selecting the fabric sheet method. In the case of application over concrete pavement, the number of reflection cracks occurring and their influence upon the drivability of motor vehicles will, for many years, be important factors in the evaluation.

REFERENCES

[1] Japan Road Association, Road Maintenance Manual, 1978 (In Japanese).

[2] Japan Road Association, Manual for Design and Construction of Asphalt Pavement, 1980, pp. 12-25.

[3] Ministry of Construction, "Study on Pavement Life and Thickness of Overlay," Proc. of Engineering Research in 1974, pp. 199-212 (In Japanese).

Table 1 Breakdowns of Public Roadways in Japan, as of April 1, 1979 (Unit: km)

	Total	Impro-ved roads	Unim-proved roads	Paved roads					Breakdown of roadway by structure		
				Total	Portland-cement-concrete pavement	Asphaltic pavement			Earthwork	bridge	tunnel
						Total	High quality pavement	Low cost pavement			
National expressways	2,430	2,430	0	2,430	108	2,322	2,322	0	2,074	291	65
Ordinary national highways	40,206	35,044	5,162	38,066	2,467	35,599	30,723	4,876	38,709	1,036	461
Prefectural roads	130,162	75,932	54,230	100,697	2,425	98,272	45,879	52,393	127,931	1,943	288
Municipal (city, town and village) roads	933,364	239,033	694,331	333,249	26,578	306,671	61,420	245,251	929,585	3,656	123
Total	1,106,162	352,439	753,723	474,442	31,578	442,864	140,344	302,520	1,098,299	6,926	937

Table 2 Length Distribution of Public Roads in Japan by Traffic Classification, as of 1979 (Unit: %)

	Road classification by traffic				
	L	A	B	C	D
One-way volume of heavy vehicle traffic (vpd)	~100	~250	~1,000	~3,000	3,000~
National (urban) expressways	0	3.6	39.0	32.4	25.0
Ordinary national highways	11.4	19.9	39.3	24.0	5.5
Prefectural roads	56.9	23.7	15.9	3.2	0.3
Subtotal	45.3	22.8	21.8	8.4	1.6
Major municipal (city, town and village) roads	78.6	12.0	7.7	1.6	0.1
Total	49.5	21.4	20.1	7.6	1.4

Note: For details on road classification by traffic volume, see Table 2.3

Table 3 Survey Extensions

Traffic class	District / Traffic volume of heavy vehicles	Ordinary district			Snowy & cold district			Total		
		Number of places	Extension	Area	Number of places	Extension	Area	Number of places	Extension	Area
Lower than Traffic B	Lower than 1,000 vehicles/day in one direction	59	140.0km	934×10³m²	66	166.5km	1,192×10³m²	125	306.5km	2,126×10³m²
Traffic C	1,000 to 3,000 vehicles	149	339.8km	2,382 "	126	310.0km	2,415 "	275	649.8km	4,797 "
Traffic D	Over 3,000 vehicles	83	191.2km	1,607 "	19	43.1km	379 "	102	234.3km	1,986 "
Total		291	671.0km	4,923 "	211	519.6km	3,986 "	502	1,190.6km	8,909 "

Table 4 Standard Maintenance Work

District / Work method	Ordinary		Snowy & cold		Total	
	Proportion of area	Cost	Proportion of area	Cost	Proportion of area	Cost
Planing only	1.0%	6 yen	0.2%	2 yen	0.7%	4 yen
Planing+surface treatment	1.5%	24 yen	1.8%	26 yen	1.6%	25 yen
Surface treatment	1.6%	18 yen	3.7%	37 yen	2.5%	26 yen
Local reconstruction	0.2%	6 yen	0.2%	4 yen	0.2%	5 yen
Total (percentage of work executed area)	4.3%	54 yen	5.9%	69 yen	5.0%	60 yen

Table 5 Total of Maintenance Costs for 10 Years

(Unit: Yen/m^2)

District / Interest rate / Traffic class	Ordinary		Snowy & cold	
	0%	7%	0%	7%
Lower than Traffic B	416 yen	471 yen	264 yen	359 yen
Traffic C	327 yen	422 yen	747 yen	1,030 yen
Traffic D	573 yen	700 yen	735 yen	1,069 yen
Whole section	456 yen	564 yen	683 yen	952 yen

Table 6 Scale of Overlay Work

	Ordinary district			Snowy & cold district		
	Overlay, 1-layer	Overlay, 2-layer	Mean	Overlay, 1-layer	Overlay, 2-layer	Mean
Mean depth of planing	4.7cm	6.9cm	5.7cm	3.1cm	3.1cm	3.1cm
Thickness of overlay	5.4cm	9.8cm	7.7cm	5.1cm	8.3cm	7.7cm
Number of years in service until overlay execution	10.4 years	10.5 years	10.5 years	10.7 years	11.9 years	11.6 years

Table 7 Standard Rehabilitation Work

		Ordinary district		Snowy & cold district	
		Area	Thickness	Area	Thickness
Overlay	1-layer	4.0%	5cm	1.4%	5cm
	2-layer	4.0%	10cm	5.7%	8cm
	Total (percentage of executed area)	8.0%		7.1%	
Planing		4.0%	5.5cm	1.4%	3cm
Number of years until execution of overlay		10.5 years		11.5 years	

Table 8 Present Serviceability Index and Corresponding Maintenance Method

Present serviceability index (PSI)	Maintenance method, outline
3 ∿ 2.1	Surface treatment
2 ∿ 1.1	Overlay
1 ∿ 0	Reconstruction

Table 9 Maintenance Criteria

Type of failure / Road classification	Rutting (mm)	Faulting (mm)		Coefficient of skid resistance	Longitudinal roughness (mm)	Percent of cracked area (%)	Diameter of pot hole (cm)
		Bridge	Culvert				
Freeway	25	20	30	0.25	8 m profilometer 90 (PrI) 3 m profilometer 3.5 (σ)	20	20
Heavy traffic road	30 ∿ 40	30	40	0.25	3 m profilometer 4.0∿5.0 (σ)	30 ∿ 40	20
Light traffic road	40	30	-	-	-	40 ∿ 50	20

Table 10 Coefficients of Relative Strength Used for Calculating T_{AO}

	Materials for existing pavement	Condition of each layer	Coefficient
Surface, binder course	Hot mix AC	Condition of failure is level 1 but tends to proceed to level 2.	0.9
		Condition of failure is level 2 but tends to proceed to level 3.	0.85∿0.6
		Condition of failure is level 3.	0.5
Base course	Hot mix stability treatment		0.8 ∿ 0.4
	Cement stability treatment		0.55∿0.3
	Lime stability treatment		0.45∿0.25
	Slag		0.55∿0.3
	Crushed stone		0.35∿0.2
Sub-base course	Crusher run, gravel		0.25∿0.15
	Cement and lime stab.		0.25∿0.15
Concrete slab	Cement concrete	Condition of failure is level 1 or 2.	0.9
		Condition of failure is level 3.	0.85∿0.5

(Note) Standard for conditions of failure

Level 1: Almost perfect performance. Maintenance and rehabilitation is not required for the time being but daily maintenance is needed (percentage of cracked area is almost less than 15%).

Level 2: Performance is almost perfect but local maintenance work is required (percentage of cracked area is between 15% and 35%).

Level 3: Large-scale maintenance and rehabilitation work such as overlay or better is required (percentage of cracked area is larger than 35%).

Table 11 Required Thickness of Overlay Based on the Deflection

(Unit: cm)

Surface deflection D (mm)	Classification of traffic volume				
	Traffic L	Traffic A	Traffic B	Traffic C	Traffic D
Less than 0.6	-	-	-	4	4
Over 0.6	-	-	4	6	8
Over 1.0	-	4	6	10	12
Over 1.5	4	6	10	12	15
Over 2.0	6	10	12	15	-

Table 12 Maintenance Criteria

Type of failure Road classification	Rutting (mm)	Faulting (mm)	Coefficient of skid resistance	Longitudinal roughness (mm)	Crack length (cm/m^2)	Failure of joint
Freeway	25	10	0.25	8 m profilometer 90 (PrI) 3 m profilometer 3.5 (σ)	20	Where any abnormality is seen
Heavy traffic road	30∿40	15	0.25	3 m profilometer 5.0 (σ)	30	
Light traffic road	40∿50	-	-	-	50	

Table 13 Outline of Concrete Pavement Surveyed

Slab thickness (cm)	Number of samples	Years to overlay	Total volume of heavy vehicles, mean (10 thousands)	Thickness of overlay (cm)
20	105	15.8	1,055	4 ∿ 20 Mean: 10
23	115	13.7	1,009	6 ∿ 23 Mean: 10

Table 14 Properties of Cut-back Blown Asphalt

Penetration (25°C)	33	Thin film, loss (%)	0.52
Softening point (°C)	96	Penetration (%)	73
		Softening point (°C)	104.0
PI	+5.1	PI	+5.2
Flash point (°C)	266	Frass BP (°C)	-19(-)
Loss on heating (%)	0.09	Ductility (20°C)	2.0
		" (15°C)	2.0
		" (10°C)	1.0
Penetration of residue (%)	95	Specific gravity (25°C/25°C)	1.20

Table 15 Performance of Overlay with Cut-back Blown Asphalt Sheet, on Concrete Pavement

No.	Location	Thickness of overlay (cm)	Year overlay	Volume of* heavy vehicles	Length of joints and cracks (m)	Length of reflection cracks, 1978 (m)	1982 Condition
1	N. Hwy 134 Nohi	10	1975	1,895	10,476	20.9	Excellent*
2	P. Road Funakoshi	3	1975	1,665	522	16	Very good, but reconstructed 1980.
3	N. Hwy 134 Tsukuihama	10	1975	1,895	2,268	3.8	Excellent*
4	N. Hwy 134 Yanagishima	13	1975	7,147	2,784	0	Excellent*
5	P. Road Ohimaizumi	10	1975	4,449	2,048	2.6	Excellent
6	N. Hwy 134 Nohi	13	1976	1,895	1,630	0	Excellent

* 1977, 12 h

** Total crack length, No. 1, No. 2 and No. 4 = 178 m, 1.2%.

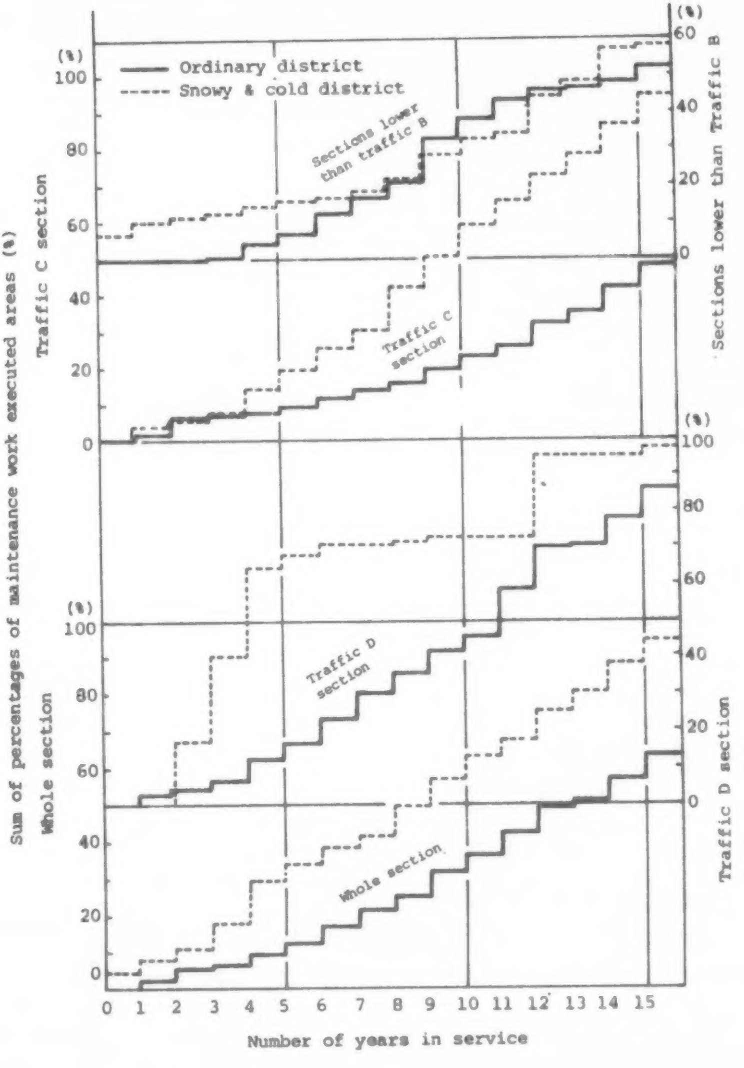

Fig. 1 Percentages of Maintenance Work Executed Areas

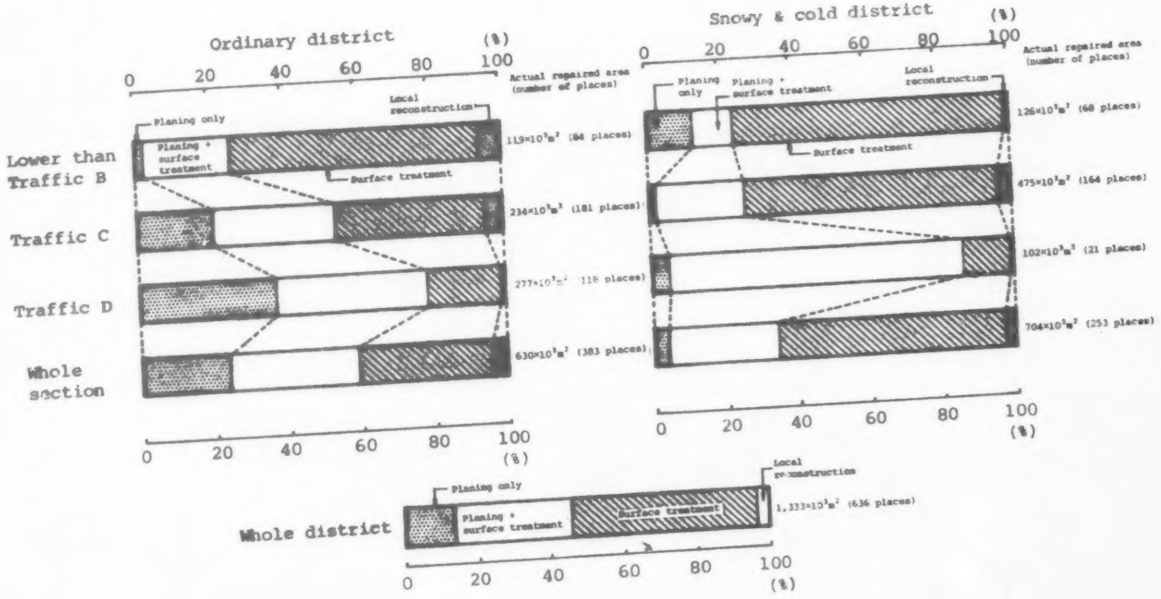

Fig. 2 Proportions of Areas by Work Method (Maintenance Work)

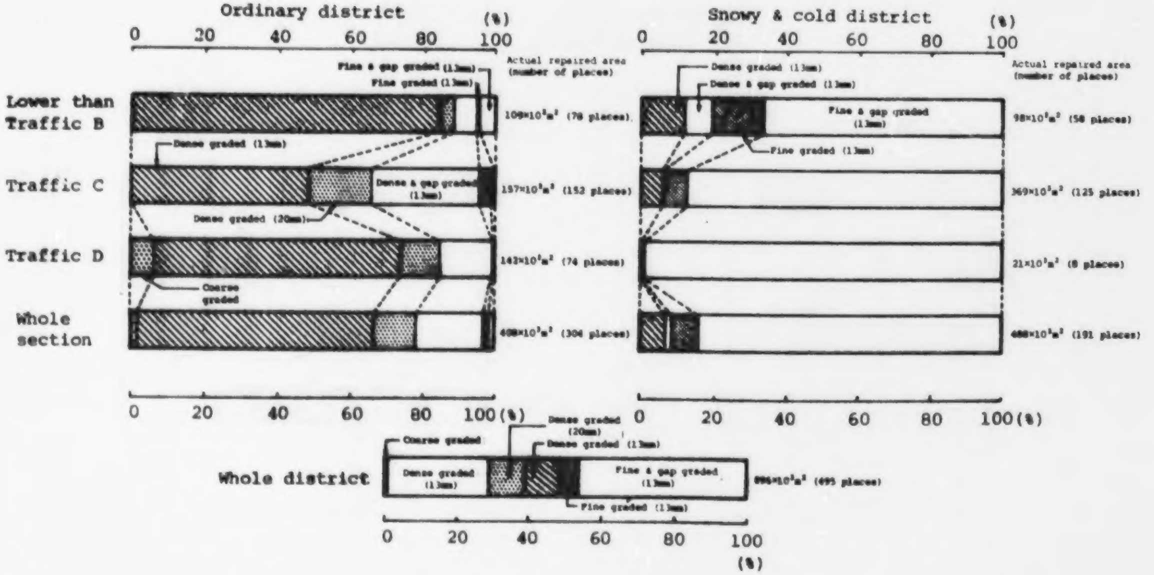

Fig. 3 Proportions of Areas by Repairing Material
(Maintenance Work)

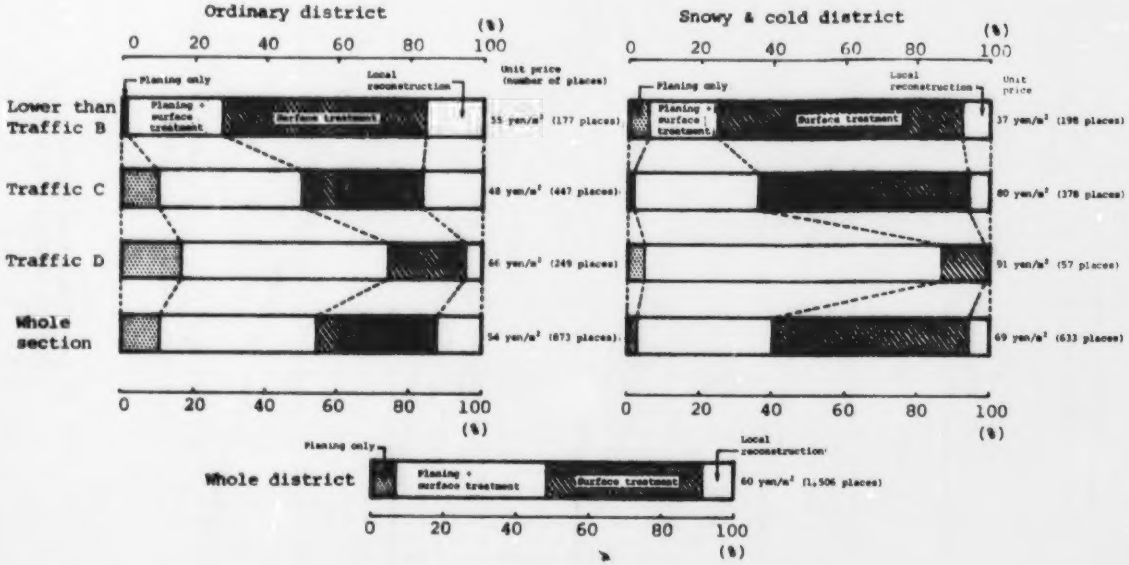

Fig. 4 Proportions of Areas by Maintenance Method
(Maintenance Work)

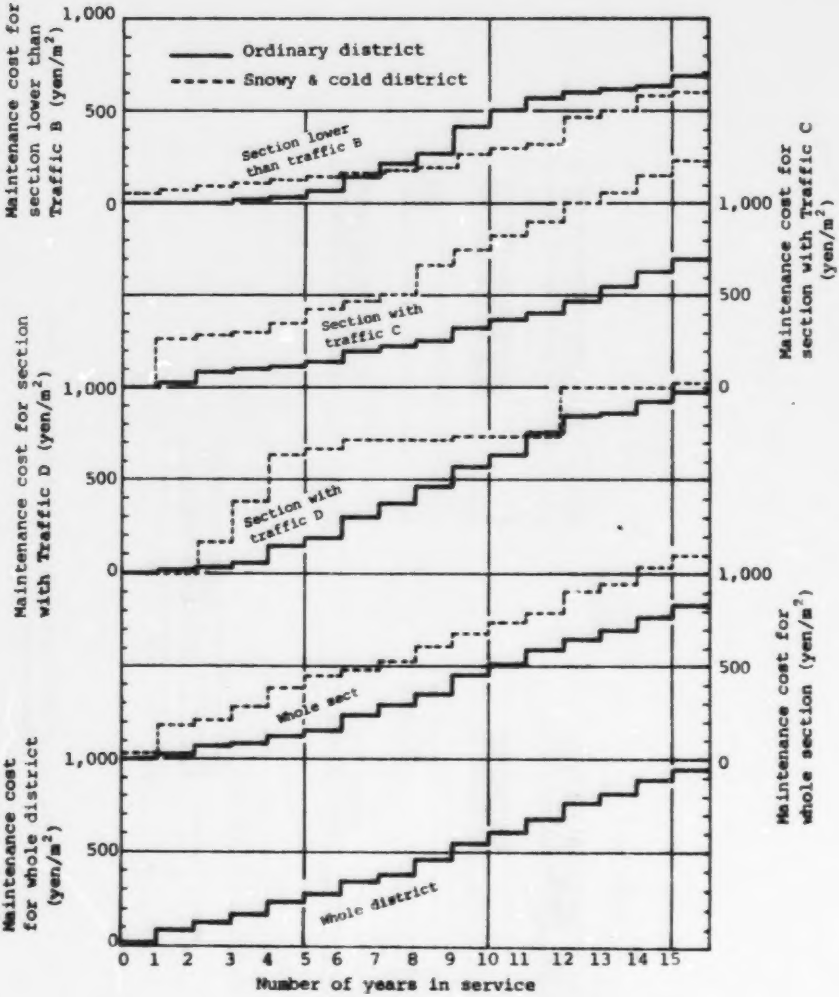

Fig. 5 Sum of Maintenance Costs (Direct Construction Costs)

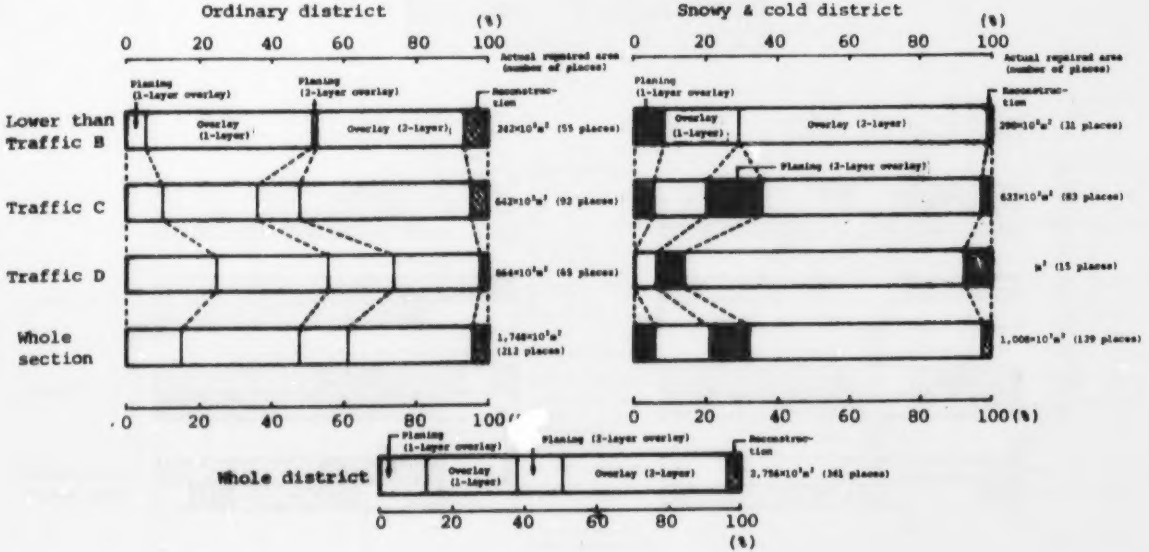

Fig. 6 Proportions of Areas by Method (Rehabilitation Work)

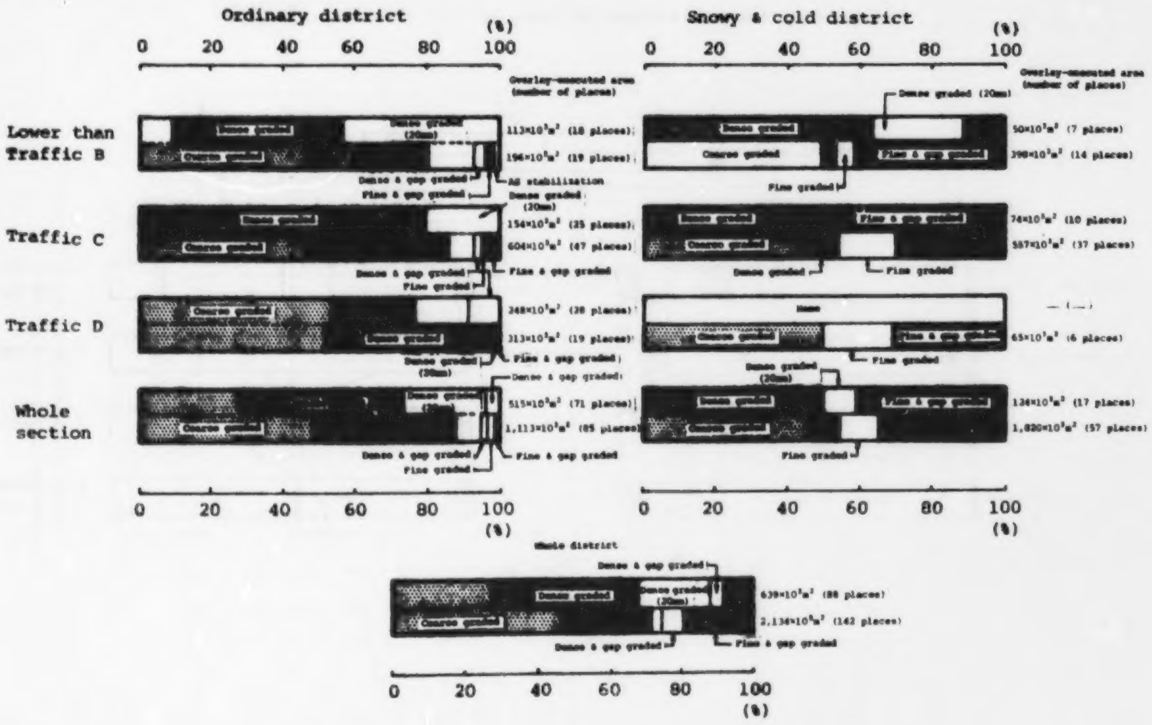

Fig. 7 Proportions of Areas by Material (Overlay Work)
(Upper row: 1-layer overlay, Lower row: 2-layer overlay)

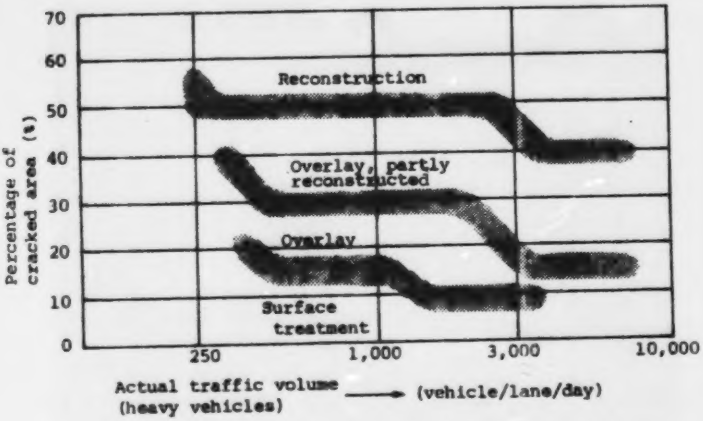

Fig. 8 Percentage of Cracked Area, Actual Traffic Volume, and Maintenance and Rehabilitation Method

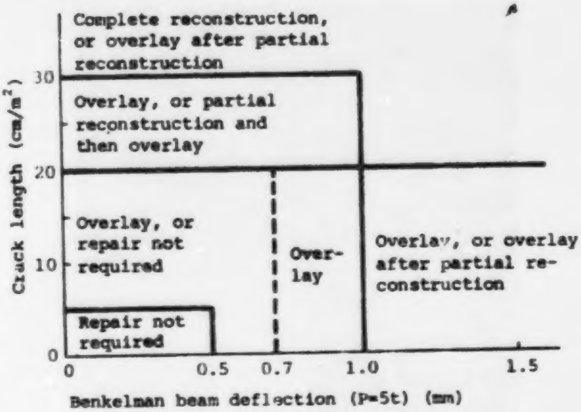
Complete reconstruction, or overlay after partial reconstruction
Overlay, or partial reconstruction and then overlay
Overlay, or repair not required
Over-lay
Overlay, or overlay after partial re-construction
Repair not required
Crack length (cm/m^2)
30
20
10
0
0
0.5
0.7
1.0
1.5
Benkelman beam deflection (P=5t) (mm)

Fig. 9 Selection of Maintenance and Rehabilitation Method on the Basis of Crack Length and Deflection

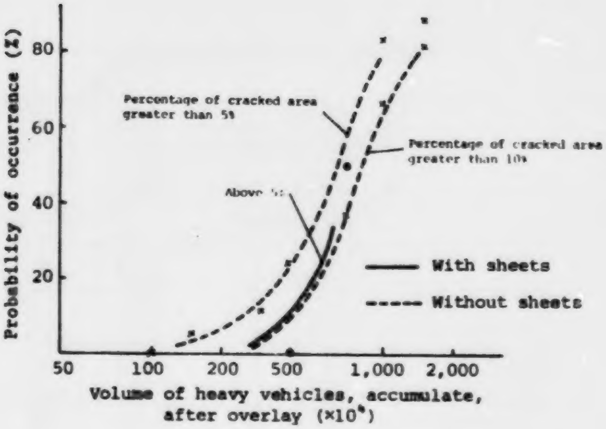

Fig. 10 Effectiveness of Use of Sheets

Photo 1 Condition of Bituminous Fabric Sheet on Concrete Pavement Joint, 1 year, No Crack

Photo 2 1 year, Reflection Cracks

Photo 3 General View of 3 cm Thin Overlay on Concrete Pavement, 7 years (Table 15, No. 2)

Photo 4 Bituminous Fabric Sheets on Asphalt Pavement Cracks, 1 year, Partially Patched

Photo 5 Bituminous Fabric Sheet on Asphalt Pavement Cracks, 7 years, Partially Patched

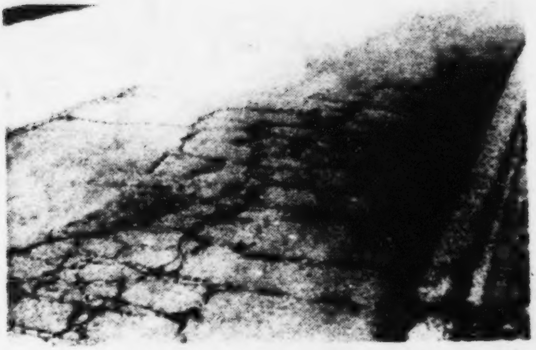

Photo 6 Cracking Condition before Overlay

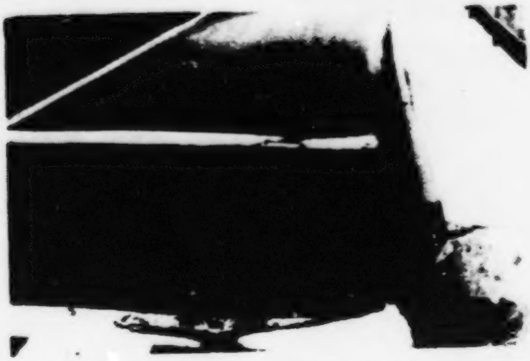

Photo 7 Cracking Condition before Overlay

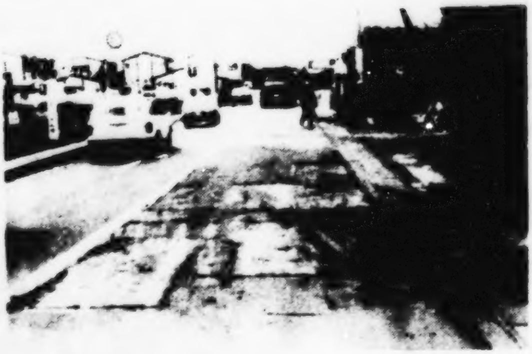

Photo 8 Bituminous Fabric Sheets on Cracks

Photo 9 Condition of 1.5 cm Thin Overlay after 7 years, Area of Photo 6

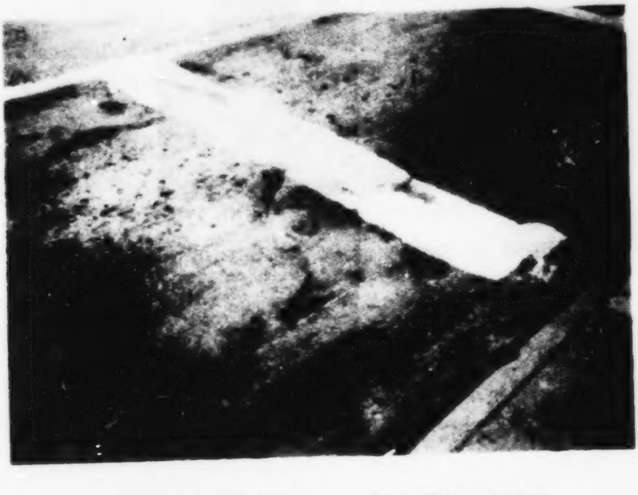

Photo 10 Condition of 1.5 cm Thin Overlay after 7 years, Area of Photo 7

Photo 11 Condition of 1.5 cm Thin Overlay after 1 year, without Fabric Sheets

Paving
in Cold
Areas

Vancouver B.C., 1982

Session 4

Overlaid Maintenance Test Section—1975

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Overlaid Maintenance Test Section - 1975

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Abstract

In the early 1970's, the Saskatchewan Department of Highways and Transportation management were evaluating procedures that would reduce the initial capital cost of new construction and rehabilitation. An overlay test section was constructed in 1975 with three (3) thicknesses, one section without an overlay, to evaluate the Benefit-Cost of each alternative. The test section was constructed on the eastbound lanes of a four-lane divided highway with an AADT of 6000 v.p.d. The test section has detailed records of construction cost and maintenance cost. Performance was measured by RCI, Benkelman Beam and Dynaflect deflections and deflection slopes, and condition surveys using aerial photography.

The paper describes the test section, summarizes the performance indicators, and gives an economic analysis covering seven (7) years. The results of the performance cost indicators follow classic pavement management models. The cost analysis shows the threshold limits of extensive maintenance by deep patching. The choice of how thick an overlay should be placed will still be made from a trade-off of pavement serviceability versus capital available for the project versus the needs of other projects.

Introduction

Prior to the sudden sharp rise in asphalt costs in the mid 1970's, the Department used asphalt concrete overlays as the main maintenance method on cracked and distorted pavements. This was done even if cracking, although extensive, was not associated with dislodgement of chunks or with distortions that significantly affected riding quality. An example is an old, age-hardened asphalt concrete extensively fatigue cracked by repeated flexure but of sufficient thickness to carry current loads without excessive deflection or shear failure.

The rapidly escalating cost of asphalt concrete caused the Department to re-examine its maintenance practices in 1974-75. One question raised was: where was the economic cut-off between spot maintenance (i.e., surface sealing, surface and deep patching, etc.) and continuous overlaying? That is, how much spot maintenance could be done on a lengthy section of pavement, maintaining acceptable levels of riding quality and aesthetics, before it would finally be more cost-effective to overlay the entire length? A test section was designed to find an answer.

In 1975, Highway 1-10 was to be resurfaced from Belle Plaine to Pense. A test section with various overlay thicknesses was incorporated into the eastbound lanes from km 28.5 to km 31.7, approximately 2 km west of Pense. The objectives were to evaluate performance of overlays of various thicknesses and to maintain a non-overlaid section to acceptable level of service. Performance was to be monitored by measurement of structural capacity and riding quality, maintenance repair and cost records, visual observations, and assessment of crack development using air photos.

Test Section Layout

The original pavement for the eastbound lanes of Highway 1-10, Regina to Moose Jaw, was laid in 1967 and consisted of 102 mm of asphalt concrete on 152 mm of granular base course and 203 mm of granular subbase course on a subgrade of lacustrine clay (unified classification CH). By 1975, the outside lane on the section from Belle Plaine east was extensively fatigue cracked and required an overlay.

A test section to evaluate performance of various thicknesses of asphalt concrete overlay and to monitor maintenance costs of non-overlaid section was incorporated into the section between Pense and Belle Plaine, specifically between km 28.5 and km 31.7. Figure 1 shows the test section layout and overlay cross-sections.

Details of the test section are:

a) each of the four test blocks is 800 m in length.

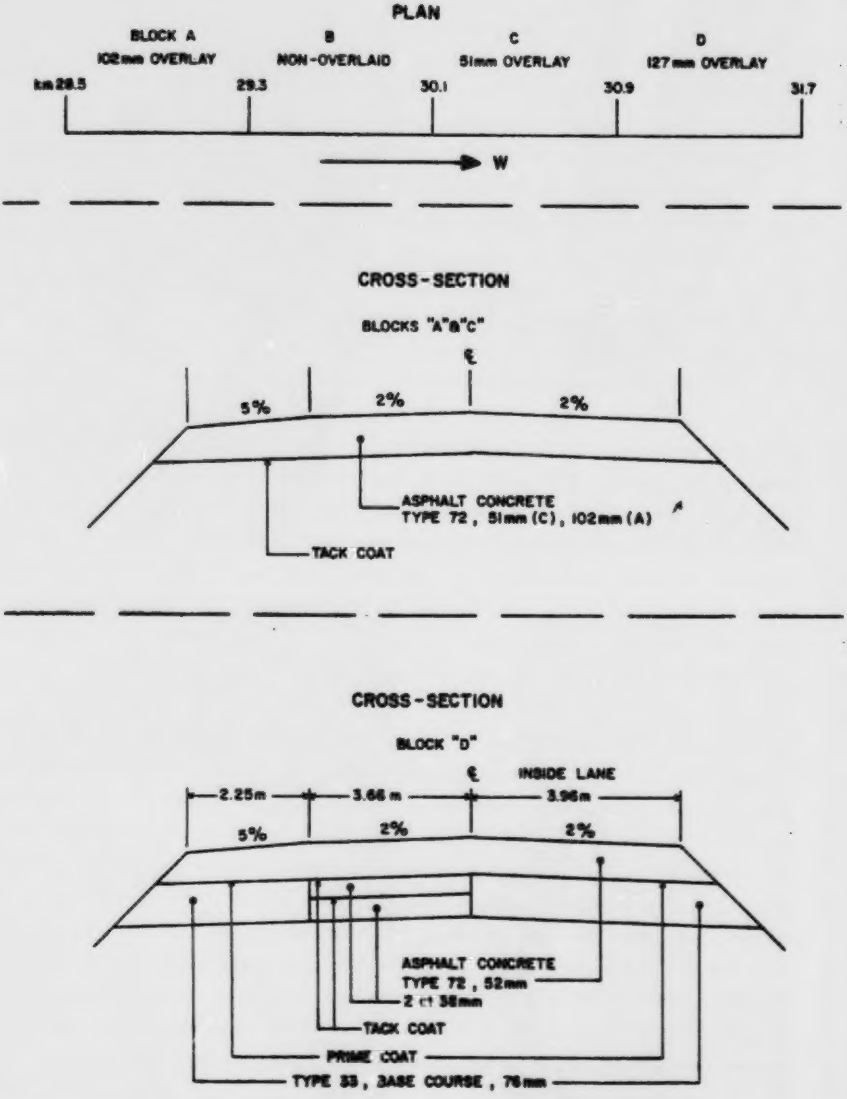
PLAN
BLOCK A
102mm OVERLAY
B
NON-OVERLAID
C
51mm OVERLAY
D
127mm OVERLAY
km 28.5
29.3
30.1
30.9
31.7
W
CROSS-SECTION
BLOCKS "A" "B" "C"
℄
5%
2%
2%
ASPHALT CONCRETE
TYPE 72, 51mm (C), 102mm (A)
TACK COAT
CROSS-SECTION
BLOCK "D"
℄
INSIDE LANE
2.25m
3.66 m
3.96m
5%
2%
2%
ASPHALT CONCRETE
TYPE 72, 52mm
2 @ 38mm
TACK COAT
PRIME COAT
TYPE 33, BASE COURSE, 76mm

Figure 1/ Details of Overlay Test Section, C.S. 1 - 10, 1975

b) all asphalt concrete is Saskatchewan Type 72 aggregate (13 mm maximum size) with Type AC 1.5 asphalt cement (approximate penetration of 383).

c) Block A is 102 mm of asphalt concrete full width.

d) Block B was not overlaid and was used for assessment of spot maintenance costs (surface sealing, surface and deep patching, etc.).

e) Block C is 51 mm of asphalt concrete full width.

f) Block D is 127 mm of asphalt concrete on the outside lane and 76 mm of Saskatchewan Type 33 granular base course (19 mm maximum size) covered with 51 mm of asphalt concrete on the inside lane.

g) all old pavement was tacked with SS-1 emulsified asphalt prior to laying new asphalt concrete on it.

Certain sections of Block B judged to need immediate attention with respect to patching in 1975 were used to establish a controlled deep patching technique to be used in subsequent years. By definition, the term 'deep patching' means replacing the full thickness of a fatigued or sheared asphalt concrete with new mix, and may include drying and recompacting underlying base course or subgrade (1). Other sections with less deterioration were given a surface seal. Approximately 119 m^2 of the outside lane were deep patched and 53 m^2 were sealed in 1975.

The deep patching technique developed for this test section is shown on Figure 2. On an old pavement with extensive transverse, longitudinal, and fatigue (alligator) cracking, dislodgement and removal is relatively simple. The full depth of 102 mm of asphalt concrete was loosened with the sharp corner of a Gradall bucket, a ripper tooth on a motor grader, or a front-end loader bucket. The excavation was then cleaned by hand shovel, the granular base recompacted, the outer edges of the base course and the walls and perimeter of the excavation were tacked with asphalt emulsion, and new AC 1.5 mix was compacted in lifts. The following day, the patch was shimmed to final elevation over the excavation and the tacked perimeter using a motor grader and vibratory compactor.

The AC 1.5 mix was used in 1975 because it was available from the main overlay job but any hot-mix can be used. For example, in 1977, a 120-150 pen mix from the City of Regina plant was used for Block B deep patching.

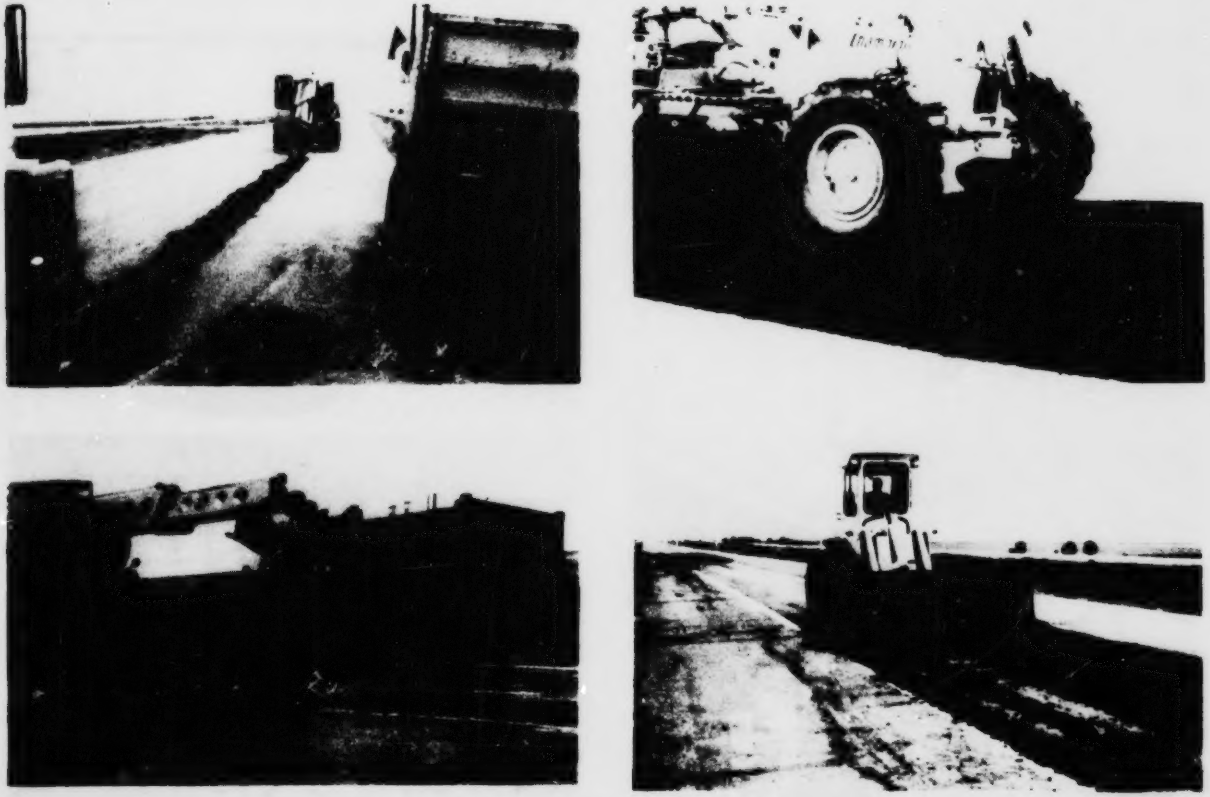

Figure 2/ Deep Patching Sequence, Overlay Test Section, C.S. 1 - 10, 1975

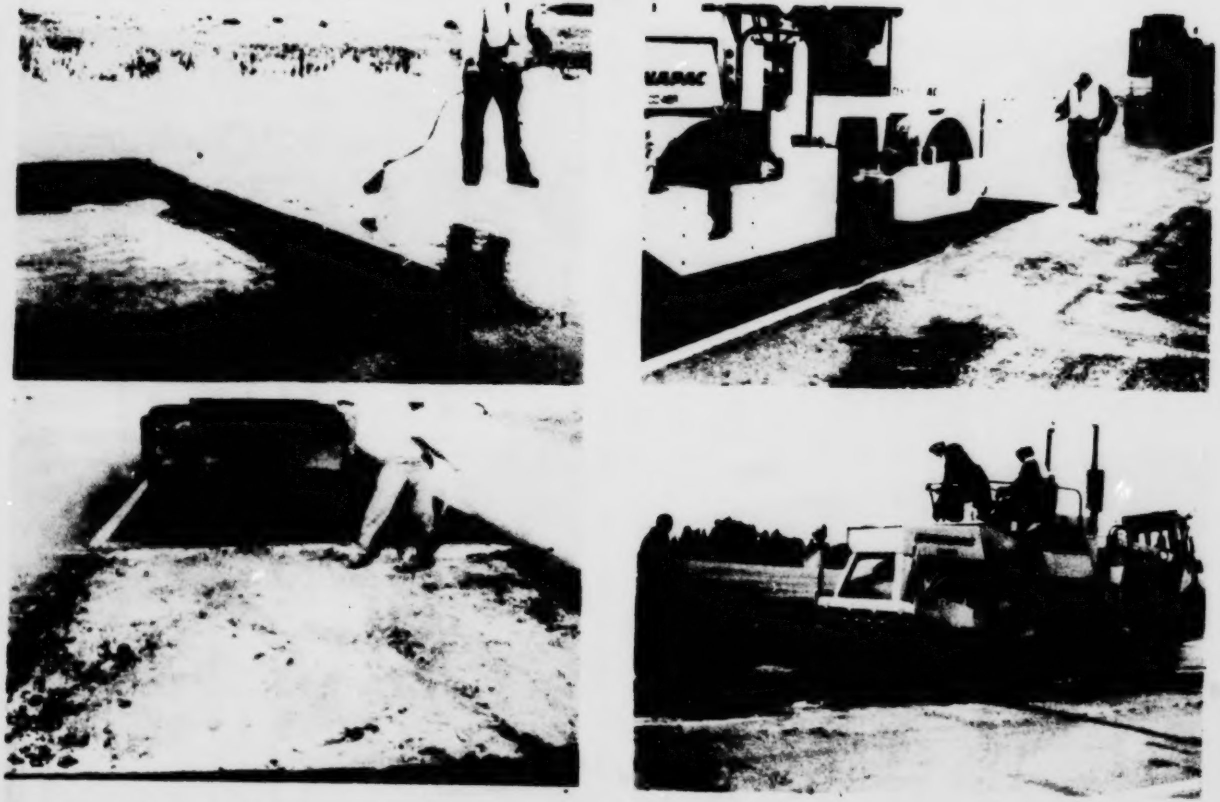

Figure 2 (Cont'd)/ Deep Patching Sequence, Overlay Test Section, C.S. 1 - 10, 1975

Performance of Overlay Sections

Performance of the four test blocks is being evaluated by surface deflections, deflection slopes, riding comfort indices, and crack development.

A. Surface Deflections and Deflection Slopes

Surface deflections were taken on the outer wheelpath of each lane annually by the standard Benkelman beam rebound method. The results are summarized in Table 1 and on Figure 3. Each value is the average of 6-7 sites on each test block. For clarity, only the values for 1976 and 1982 are plotted on Figure 3. Also plotted are the 1975 values for the outside lane of the old pavement prior to overlaying. (The exaggerated vertical scale of Figure 3 distorts the picture somewhat.)

TABLE 1

AVERAGE DEFLECTIONS, BENKELMAN BEAM METHOD, mm

BLOCK	LANE	OVERLAY THICKNESS	1975	1976	1977	1978	1979	1980	1981	1982
A	Outside	102 mm	1.36	0.86	0.82	0.83	0.73	0.75	0.93	0.60
A	Inside	102	-	1.20	1.13	1.18	1.13	1.20	1.22	1.07
B	Outside	0	1.42	1.17	1.31	1.41	1.10	1.27	1.41	0.85
B	Inside	0	-	1.86	1.67	1.94	2.13	1.78	1.99	1.19
C	Outside	51	1.29	0.92	0.80	0.90	0.74	0.84	1.00	1.27
C	Inside	51	-	1.29	1.28	1.32	1.47	1.28	1.34	1.89
D	Outside	127	1.32	0.73	0.64	0.71	0.57	0.61	0.74	0.72
D	Inside	127	-	0.93	0.95	1.00	1.05	0.98	1.00	1.14

The data shows:

a) a decrease in deflection with the addition of an overlay, the magnitude of the change increasing as overlay thickness increased. For the outside lane, the changes through the first year were 29, 37 and 45% for 51, 102, and 127 mm thicknesses, respectively. By 1982, deflection on the 51 mm block had regained the 29% decrease whereas the thicker blocks were still maintaining the improvements.

b) higher deflections on the inside lane than on the outside lane, the magnitude of the difference lessening as overlay thickness

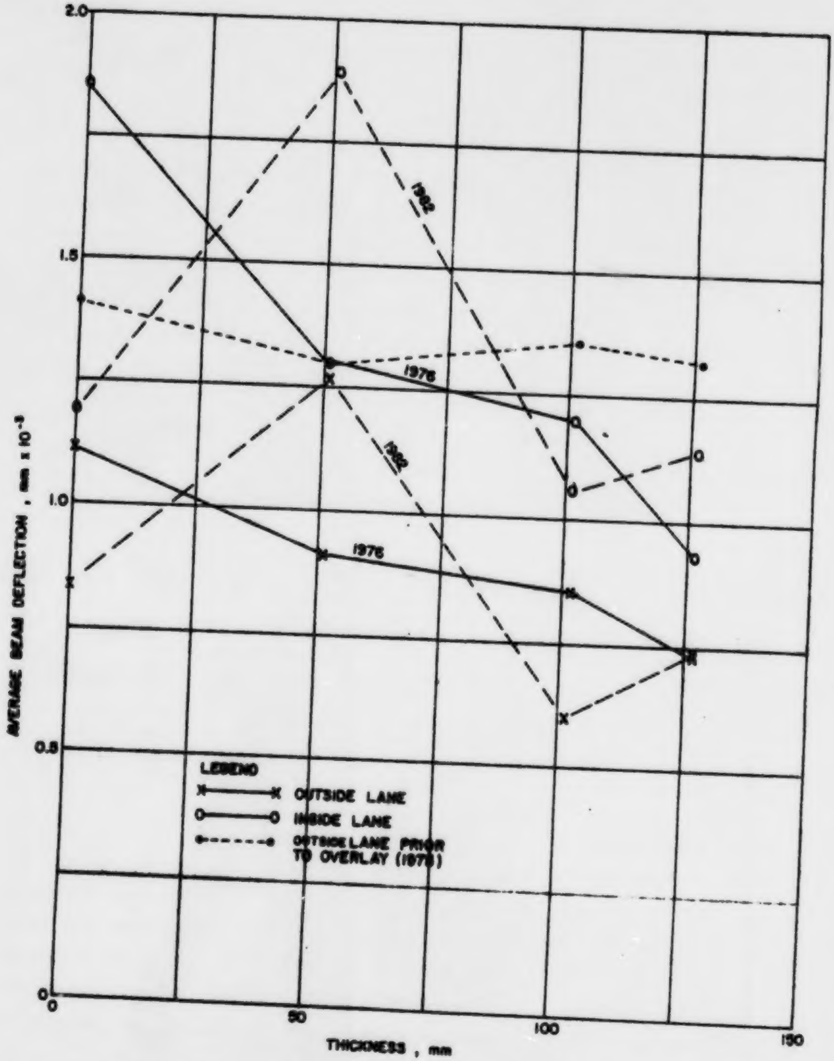

Figure 3/ Average Beam Deflections, Overlay Test Section, C.S. 1 - 10 increased. The cause of this difference is probably because the outer wheelpath of the inside lane has approximately 2 m less lateral support (narrower shoulder) than that of the outside lane. Deflection patterns by 1982 had followed those of the outside lane.

c) Although the difference in deflection between the 127 mm asphalt concrete on the outside lane of block D and the 127 mm sandwich structure on the inside lane is not yet significant, a trend of higher deflection on the latter appears to be forming.

Deflection slopes also were measured annually by beam and in 1980 by Dnyaflect. Deflection slope is equivalent to radius of curvature of a deflection basin. It is the linear slope between the deflection at the centre of the loaded area and at a point 30 cm away from it. The higher the value, the sharper the radius and the higher the stresses due to bending or flexure.

Table 2 and Figure 4 summarize the beam deflection slope data. Each point is the average of two sites. For clarity, only the values for every second year are plotted. Generally, slopes decreased as overlay thickness increased, the effect becoming more distinct with time. This is probably due to increasing stiffness of asphalt binder with time.

TABLE 2

AVERAGE DEFLECTION SLOPES x 10^{-3}

BLOCK	LANE	OVERLAY THICKNESS	1976	1977	1978	1979	1980	1981	1982
A		102 mm	8.0	-	6.2	9.5	5.5	7.2	3.8
B		0	13.0	-	20.3	16.8	10.3	14.8	12.0
C		51	8.5	-	9.7	10.3	6.3	9.3	6.0
D	Outside	127	8.8	-	9.3	7.8	3.8	8.0	3.7
D	Inside	127	13.8	-	15.8	16.8	9.3	10.3	13.0

NOTE: 1977 DATA ERRONEOUS AND NOT SHOWN

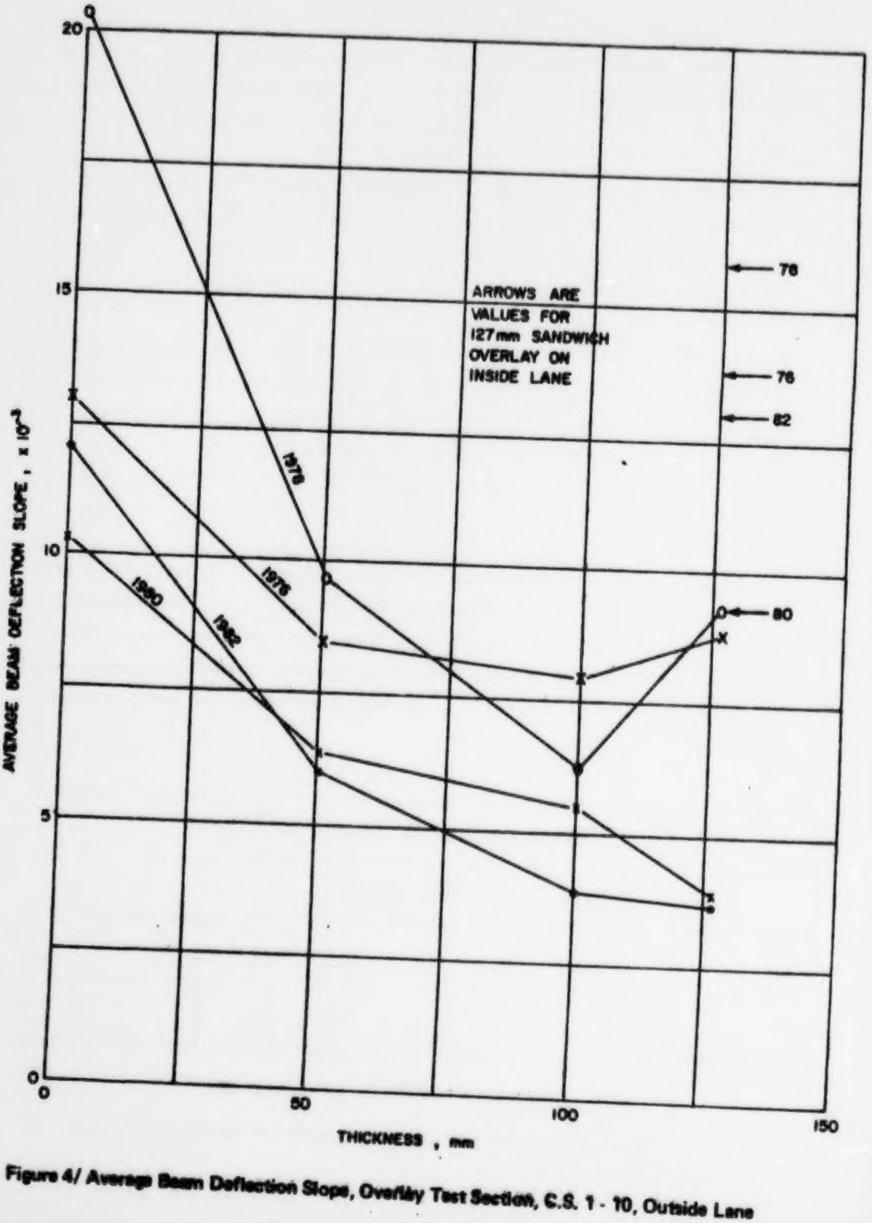

Figure 4/ Average Beam Deflection Slope, Overlay Test Section, C.S. 1 - 10, Outside Lane

Similar to peak deflections, deflection slopes for the 127 mm sandwich structure on the inside lane of block D were significantly higher than for the adjacent full depth asphalt concrete. In 1982, the difference was almost four times, indicating sharper bending or flexural curvature and higher stresses and strains. To date, no difference in surface condition is evident but it is logical to expect such defects as rutting, shear, or washboarding to occur on the inside lane before the outside lane.

In 1980, Dynaflect deflections and deflection slopes were measured on the same sites as the beam tests. The data is shown in Figure 5. Each point is the average of two sites. The magnitudes of the values differ from the beam values but the trends are similar, with values decreasing with increasing thickness and lower on the outside lane than on the inside lane.

B. Riding Quality (Riding Comfort Index)

For measurement of pavement riding quality, the Department uses a standard sedan instrumented to measure movement of its rear axle. These movements are converted into a "riding comfort index (RCI)" from 0 - 10, the higher the value the better the ride. For new pavements and overlays of asphalt concrete, the Department objective is 8.0. When the level has dropped to 5.5, an overlay is usually applied if other conditions are also in a critical state. Very few pavements are overlaid for ride alone. RCI values would be between 5.0 and 5.5 at time of overlay.

Table 3 and Figure 6 summarize index data for six years, 1977-1982 inclusive. None of the new overlays attained the desired RCI of 8.0 immediately after construction.

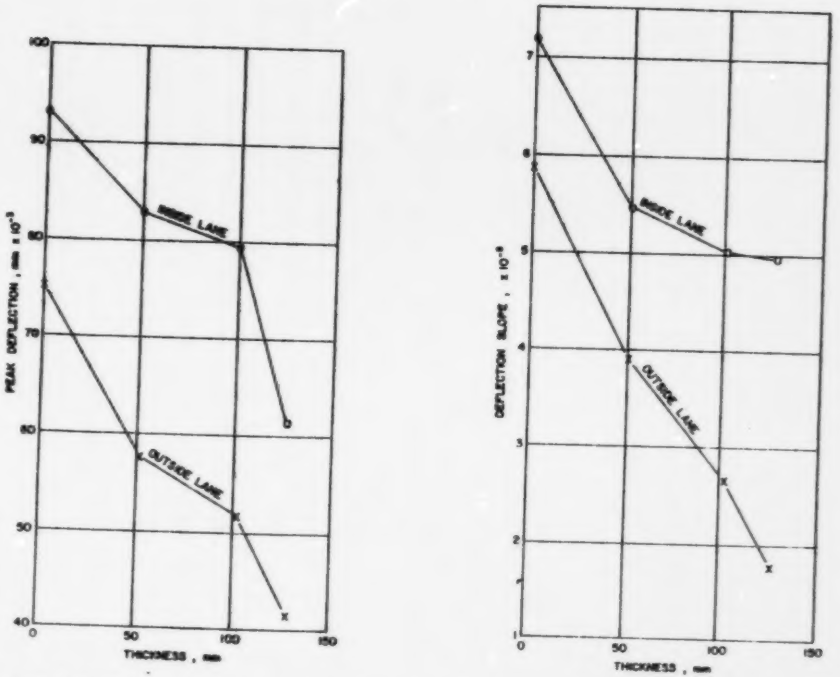

Figure 5/ Dynaflect Peak Deflections and Deflection Slopes, Overlay Test Section, C.S. 1 - 10, June 12/80

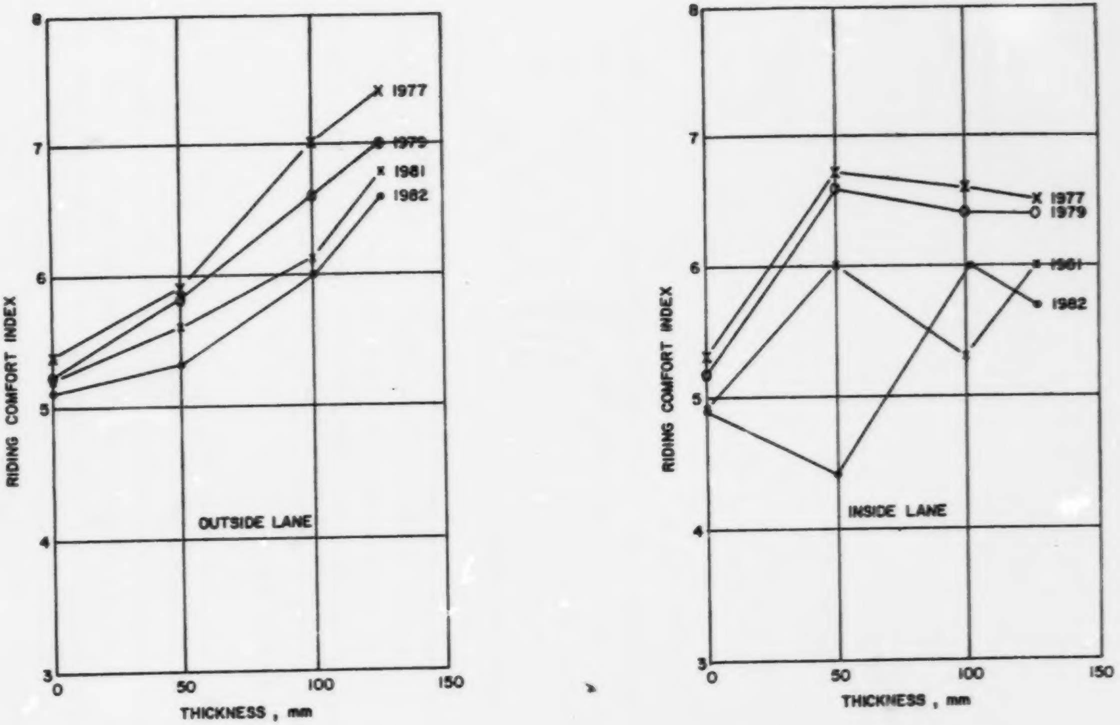

Figure 6/ Riding Comfort Indices, Overlay Test Section, C.S. 1 - 10

TABLE 3

RIDING COMFORT INDICES

BLOCK	LANE	OVERLAY THICKNESS	1977	1979	1981	1982
A	Outside	102 mm	7.0	6.6	6.1	6.1
	Inside		6.6	6.4	5.3	6.0
B	Outside	0	5.4	5.2	5.2	5.1
	Inside		5.3	5.1	4.9	4.9
C	Outside	51	5.9	5.8	5.6	5.3
	Inside		6.7	6.6	6.0	4.4
D	Outside	127	7.4	7.0	6.8	6.6
	Inside		6.5	6.4	6.0	5.7

For the outside lane, RCI improved as overlay thickness increased, from approximately 5.4 before overlaying to 7.4 with addition of 127 mm of new asphalt concrete. Six years later, in 1982, RCI had dropped to 6.6. Patterns for the thinner overlays were similar but the values were proportionately less.

In the same six year period, the RCI for Block B, the non-overlaid section, dropped from 5.4 to 5.1 on the outside lane. This indicates that the extensive maintenance activities (see following section of Paper) of spot sealing and deep patching are holding the section at its 1977 level of service. The cost effectiveness of this practice is discussed later.

The RCI patterns for the inside lane were more erratic. There was an improvement from 5.3 to 6.7 by addition of 51 mm of asphalt concrete but no further improvement with the thicker structures. The drop in RCI over the 6-year period for the 51 mm section, to 4.4, was much larger than for the outside lane; for the 127 mm section, the amount of loss matched that of the outside lane.

By 1982, the 51 mm section was below the RCI level of 5.5. This section has deteriorated to the level of the non-overlaid section.

C. Crack Development

Crack development was monitored by means of air photos taken prior to overlaying and again in 1976 and 1978. (Another run will be made in mid-1982) Transverse and longitudinal cracks were either freshly sealed with asphalt or painted with black enamel for contrast on the photos; fatigued or sheared areas were either spot sealed or area-outlined with enamel. The primary purpose of these photos was to monitor the effectiveness of overlays in preventing or delaying reflective cracking. A secondary reason was to have a permanent record of exactly where and how much spot sealing and deep patching was done on Block B annually.

Table 4 summarizes the development of transverse cracking on the overlaid sections. The cracks are classified as crossing both lanes or one lane only. Figure 7 shows a typical 61 m section of Block C, the 51 mm overlay, and Figure 8 a typical 61 m section of Block D, the 127 mm section. (These sketches were transferred from the air photos for simplicity of copying for this Paper).

TABLE 4

DEVELOPMENT OF TRANSVERSE CRACKS

BLOCK	LANE	OVERLAY THICKNESS	NUMBER OF CRACKS 1976	1978
A	Outside	102 mm	3	8
	Inside		7	8
	Both		32	38
B		0	-	-
C	Outside	51	9	12
	Inside		8	14
	Both		73	76
D	Outside	127	5	9
	Inside		2	3
	Both		47	34*

NOTES:
1. * 180 m of 1978 photos for Block D missing from run.
2. Block B too extensively alligatored to separate transverse cracks.

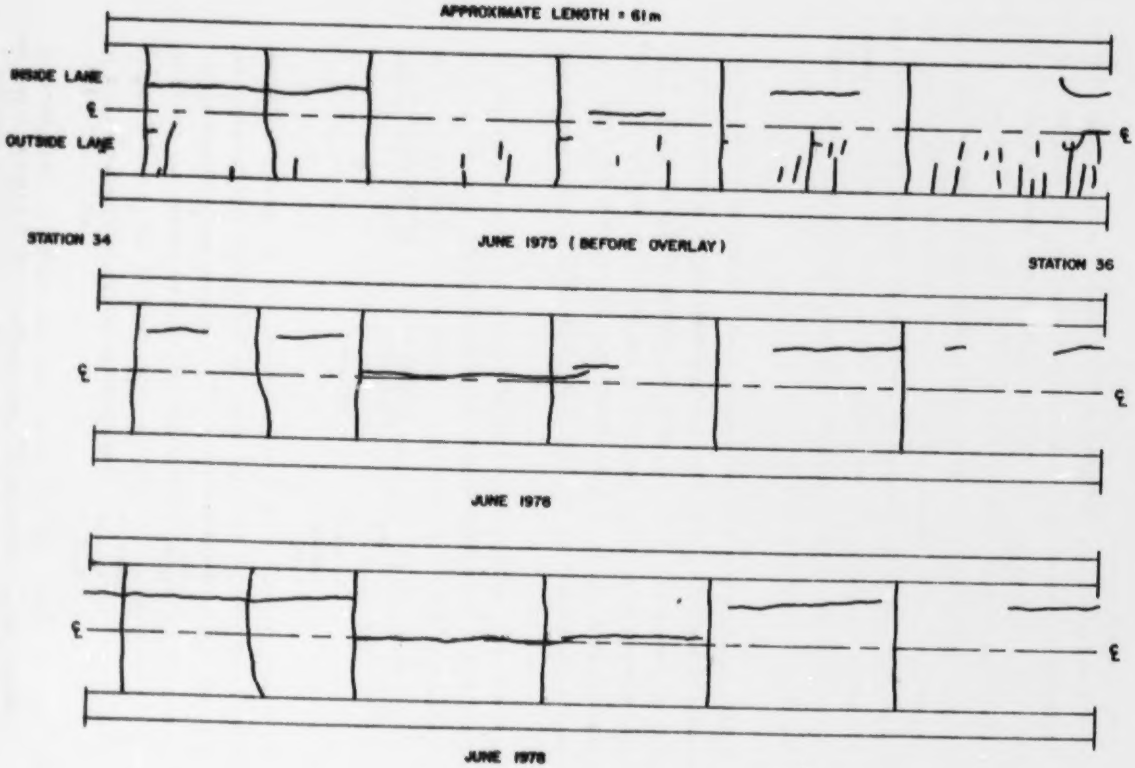
APPROXIMATE LENGTH = 61m
INSIDE LANE
℄
OUTSIDE LANE
℄
STATION 34
JUNE 1975 (BEFORE OVERLAY)
STATION 36
℄
℄
JUNE 1976
℄
℄
JUNE 1978

Figure 7/ Crack Patterns, Block C, 51 mm Overlay, Overlay Test Section, C.S. 1 - 10

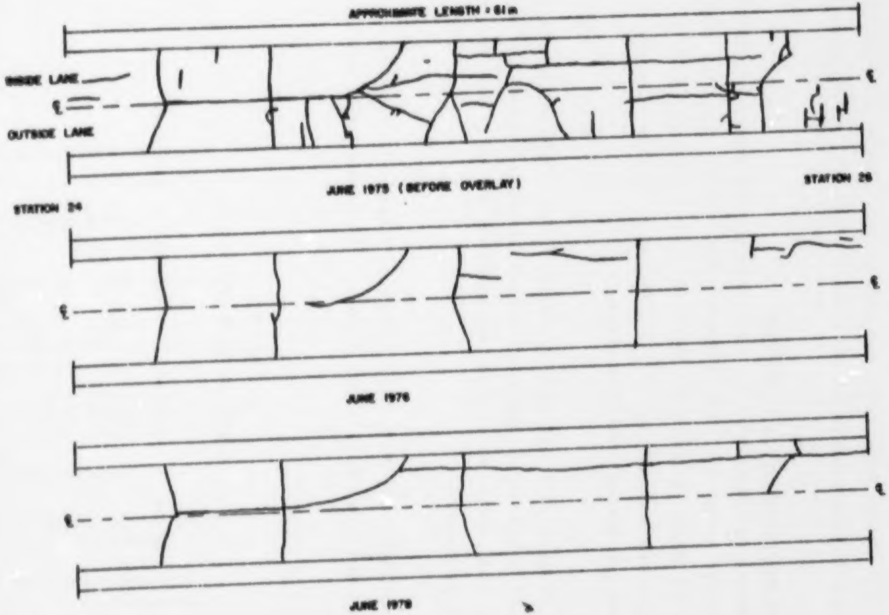
APPROXIMATE LENGTH = 61 m
INSIDE LANE
℄
OUTSIDE LANE
℄
STATION 26
JUNE 1975 (BEFORE OVERLAY)
STATION 24
℄
℄
JUNE 1976
℄
℄
JUNE 1979

Figure 8/ Crack Patterns, Block D, 127 mm Overlay, Overlay Test Section, C.S. 1 - 10

The data shows without doubt that the overlay structures used on this test section, whether asphalt concrete or sandwiched granular base course, neither prevented nor delayed reflection of cracking regardless of thickness of the overlay. This has been shown many times, in Saskatchewan and elsewhere, but simply overlaying still remains a favorite method of treating a badly cracked pavement.

Maintenance Activities On Test Section

The maintenance activities on all four test blocks from the completion of overlay construction in 1975 through 1981 were recorded, sample of recordings is shown in Table 5. Spot sealing and deep patching done on the old pavement as part of the overlaying operation are not shown as "maintenance activities" for this purpose.

The table is split into two parts, one of visual observations of surface condition as noted by a Research Branch technician and the other of activities and costs of actual maintenance as provided by District maintenance personnel.

Prior to overlaying in 1975, deep patching of badly fatigued areas of the outside lane was done on all four blocks. The amounts done were as shown in Table 6.

TABLE 6

AREA OF DEEP PATCHING

Block	Area m^2	Percent of Lane Area
A	110	3.8
B	153	5.2
C	177	6.0
D	120	4.1

Maintenance on the three overlaid blocks after 1975 consisted only of crack sealing with asphalt sealant and blotter sand. Over the 6-year period, the amounts of asphalt used in 51 mm, 102 mm, and 127 mm overlays, respectively, were 1672, 1486 and 1389 litres. Total costs were $1674, $1971 and $1463.

TABLE 5

SURFACE MAINTENANCE RECORD

TEST SECTION: CS 1-10

DATE	RESEARCH BRANCH OBSERVATIONS	DATE	DISTRICT MTCE. ACTIVITY	AGGREGATE TONNE	ASPHALT LITRE	EQUIPMENT AND LABOUR COST
SECTION A:	km 28.5 to km 29.3, 102 mm AC-5 Overlay, Full Width					
75-7-29	Construction was completed					
		76-5-21	crack seal	2	114	101.46
76-11-16	Transverse cracks 6 mm wide excellent ride, appearance, and performance					
77-4-18	No deterioration, only crack filling required					
		77-5-3	crack seal	2	182	857.59
78-3-29	No deterioration					
		78-5-3	crack seal	10	227	148.28
79-4-3	Transverse cracks 6-18 mm wide; condition unchanged					
		79-6-5	crack seal	10	227	276.94
80-5-1	Performance unchanged; spalling not as apparent as in blocks C & D					
		80-6-11	crack seal	1	168	83.18
80-9-25	Good appearance; transverse cracks causing some riding deterioration					$1,467.45

The block of primary interest with respect to maintenance activities and costs was Block B, the non-overlaid section. The intention here was to hold the section to an acceptable level of service by maintenance, rather than by overlaying, until it was no longer a cost effective practice. Maintenance consisted of crack sealing, spot surface sealing, and deep patching. In the 6-year period 1976-1981, 1594 m^2 or 54.4% of the outside lane were deep patched and another 322 m^2 or 11.0% were sealed. Total cost for this work and crack sealing was $47,221. By the end of 1981, approximately 65.4% of the outside lane area had received major maintenance; the inside lane had received only crack sealing.

Economic Analysis

The objective of the overlay was to verify performance and economics of standard overlay, substandard overlay and extensive maintenance options. Block B is the only section that is at or beyond one life cycle (service life) (2), the other sections have yet to reach their first life cycle. A prediction method or equalization process has to be used to do an economic comparison on the four sections. The method chosen was to design and estimate cost of an overlay to equalize the future service life of all of the sections.

The required overlay thickness for each section is summarized in Table 7, established using the Department's standard procedure (3). The cost of overlay to equalize the life cycle on all test blocks is summarized in Table 8. There is nothing unexpected in the cost to return all sections to an equal service life. The thickest and most costly rehabilitation is required on Block B, the non overlaid section, and the least costly is on Block D, the 127 mm overlay.

TABLE 7

SUMMARY OF DESIGN THICKNESSES AND RECOMMENDED OVERLAY

		DEFLECTION METHOD	SASKATCHEWAN METHOD				
SECTION	LANE	OVERLAY THICKNESS	MINIMUM	MAXIMUM	SURFACE[1] CONDITION	OVERLAY THICKNESS	RECOMMEND OVERLAY (mm)
A	Outer	95	0	5	2.5	3	95
A	Inner	70	0	0	2.5	0	95
B	Outer	140	58	85	3.5	77	140
B	Inner	90	0	0	3.5	0	140
C	Outer	105	7	45	2.0	22	105
C	Inner	75	0	0	2.0	0	105
D	Outer	60	0	0	1.0	0	60
D	Inner	35	0	0	1.0	0	60

[1] Surface condition is estimate only.

The profile cross-section of all the blocks is shown in Figure 9. The final thicknesses required do not end up at the same elevation as design criteria is largely dependent on fatigue considerations.

Present Worth Analysis

The economic analysis used to bring all costs to 1982 values uses Canada Pension Plan Debentures (4) and the construction price index for Saskatchewan from Statistics Canada. The cash flow for all blocks is shown in Table 9, using the interest rate of Canada Pension Plan Debentures (CPPD) to calculate the present worth of all the expenditures to date. This calculation assumed funds were borrowed at the time this work was started. The accumulated present worth using this method is shown in Figure 10 and shows that Block B (the maintenance section) has the highest accumulated cost and requires the highest input of funds to bring it up to standard. Block C (51 mm overlay) has the lowest requirement of funds.

TABLE 8

ESTIMATED COSTRUCTION COSTS PER KM FOR BLOCK OVERLAYS

ITEM[1]	TEST SECTION A		TEST SECTION B		TEST SECTION C		TEST SECTION D	
	QUANTITY	COST	QUANTITY	COST	QUANTITY	COST	QUANTITY	COST
Hot Mix Surf. Crse.	910	21 321	1 461	34 231	1 070	25 070	565	13 238
Haul of Hot Mix	22 300	3 122	34 000	4 760	24 500	3 430	14 300	2 002
Sub-Total		24 443		38 991		28 500		15 240
Tack Coat	10 310	1 753	11 590	1 970	11 000	1 870	9 880	1 680
Binder and Filler	45	135	73	219	54	162	28	84
Repair Failures	-	-	-	1 000	-	-	-	-
Force Account	-	1 500	-	1 500	-	1 500	-	1 500
Traffic Accom.	2.5% S.T.	611	2.5% S.T.	975	2.5% S.T.	713	2.5% S.T.	381
Sundries	0.5% S.T.	122	0.5% S.T.	195	0.5% S.T.	143	0.5% S.T.	76
Engineering	5% S.T.	1 222	5% S.T.	1 950	5% S.T.	1 425	5% S.T.	762
Total/km		29 786		46 800		34 312		19 723

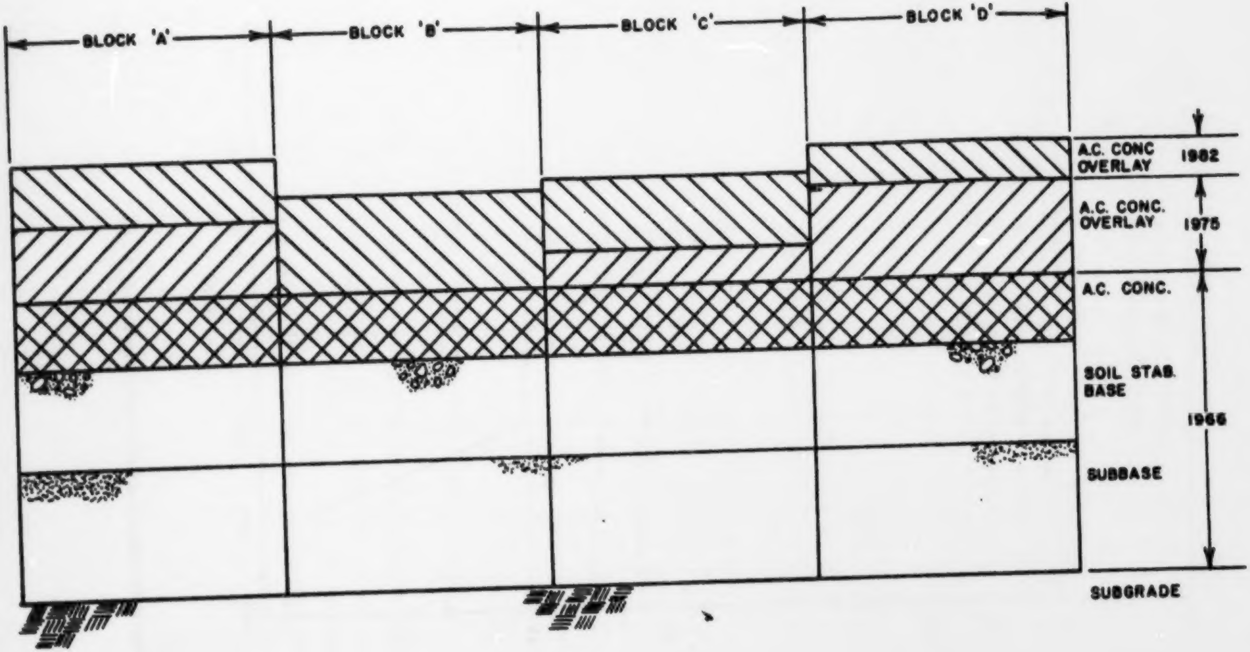
BLOCK 'A'
BLOCK 'B'
BLOCK 'C'
BLOCK 'D'
A.C. CONC OVERLAY
1982
A.C. CONC. OVERLAY
1975
A.C. CONC.
SOIL STAB. BASE
1966
SUBBASE
SUBGRADE

Figure 9/ Test Section Structural Thickness

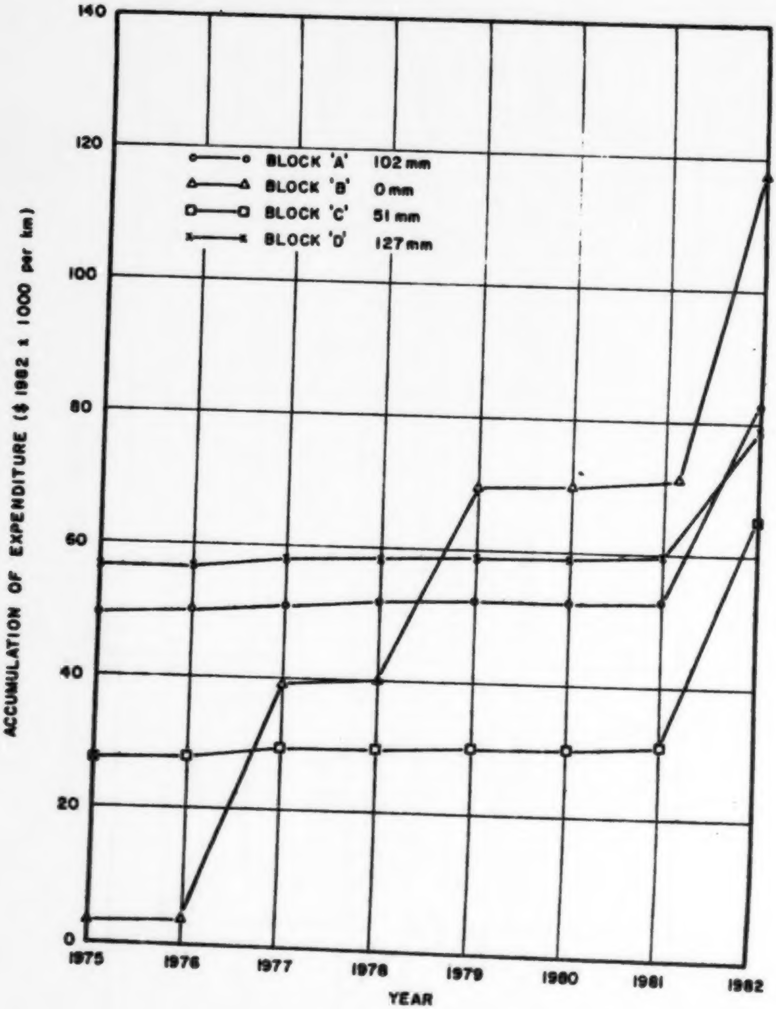

Figure 10/ Debenture Cost Present Worth, C.S. 1 - 10

TABLE 9

CASHFLOWS FOR TEST SECTION

USING DEBENTURE RATES

	BLOCK 'A'				BLOCK 'B'			
YEAR	CAPITAL $/km	MAINT. $/km	TOTAL $/km	1982 COST	CAPITAL $/km	MAINT. $/km	TOTAL $/km	1982 COST
1975	26 633	1 396	28 029	49 808	-	1 940	1 940	3 447
1976	-	148	148	249	-	143	143	241
1977	-	1 106	1 106	1 687	-	23 350	23 350	35 609
1978	-	257	257	368	-	598	598	856
1979	-	442	442	592	-	21 896	21 896	29 341
1980	-	141	141	178	-	550	550	696
1981	-	360	360	407	-	684	684	774
1982	29 786	-	29 786	29 786	46 800	-	46 800	46 800
			60 269	83 075			95 411	117 763

	BLOCK 'C'				BLOCK 'D'			
YEAR	CAPITAL $/km	MAINT. $/km	TOTAL $/km	1982 COST	CAPITAL $/km	MAINT. $/km	TOTAL $/km	1982 COST
1975	13 409	2 245	15 654	27 817	30 433	1 521	31 954	56 782
1976	-	143	143	241	-	98	98	165
1977	-	831	831	1 267	-	831	831	1 267
1978	-	129	129	185	-	129	129	185
1979	-	377	377	505	-	377	377	505
1980	-	366	366	463	-	148	148	187
1981	-	246	246	278	-	246	246	278
1982	34 312	-	34 312	34 312	19 723	-	19 723	19 723
			52 058	65 068			53 506	79 092

The use of construction price to calculate the present worth gives similar results (Table 10 and Figure 11) to the use of interest rates calculated from Canada pension plan debentures. This shows that either discount rate can be used without changing the outcome of the results.

TABLE 10

CASHFLOW FOR TEST SECTION USING

HOT MIX CONSTRUCTION PRICE INDEX

	BLOCK 'A'		BLOCK 'B'		BLOCK 'C'		BLOCK 'D'	
YEAR	EXPEND $/km	1982 COST	EXPEND $/km	1982 COST	EXPEND $/km	1982 COST	EXPEND $/km	1982 COST
1975	28 029	57 740	1 940	3 996	15 654	32 247	31 954	65 825
1976	148	314	143	303	143	303	98	208
1977	1 106	2 014	23 350	42 520	831	1 513	831	1 513
1978	257	435	598	1 012	129	218	129	218
1979	442	620	21 896	30 720	377	529	377	529
1980	141	195	550	760	366	506	148	205
1981	360	425	684	807	246	290	246	290
1982	29 786	29 786	46 800	46 800	34 312	34 312	19 723	19 723
		91 529		126 918		69 918		88 511

Another question that is quite often asked is - would the savings by using a thinner overlay finance future fund requirements? To analyze this, a comparison of the cost of all the alternatives to Block D is summarized in Table 11. Using the CPPD interest rate, as the investment cost, the cost to upgrade all blocks requires $22,925, $57,614, $-955, $19,723 for blocks A, B, C and D respectively. This again shows that block C is an economic means of rehabilitation when considering the capital cost of a project.

A calculation was performed to calculate the breakeven point of deep patching versus overlay. The test Block B had 65.4% of lane area deep patched. A comparison to cost of Block C shows that about 6.8% of the outside lane can be maintained using a deep patching program. This value relates to the general observation that the level of 5% failure in a pavement surface is where maintenance personnel sound the alarm that they need an overlay.

Conclusions

The overlay test section presents conclusions that can be used by the Department at this time, further conclusions will be available at the terminal lives of Blocks A and D.

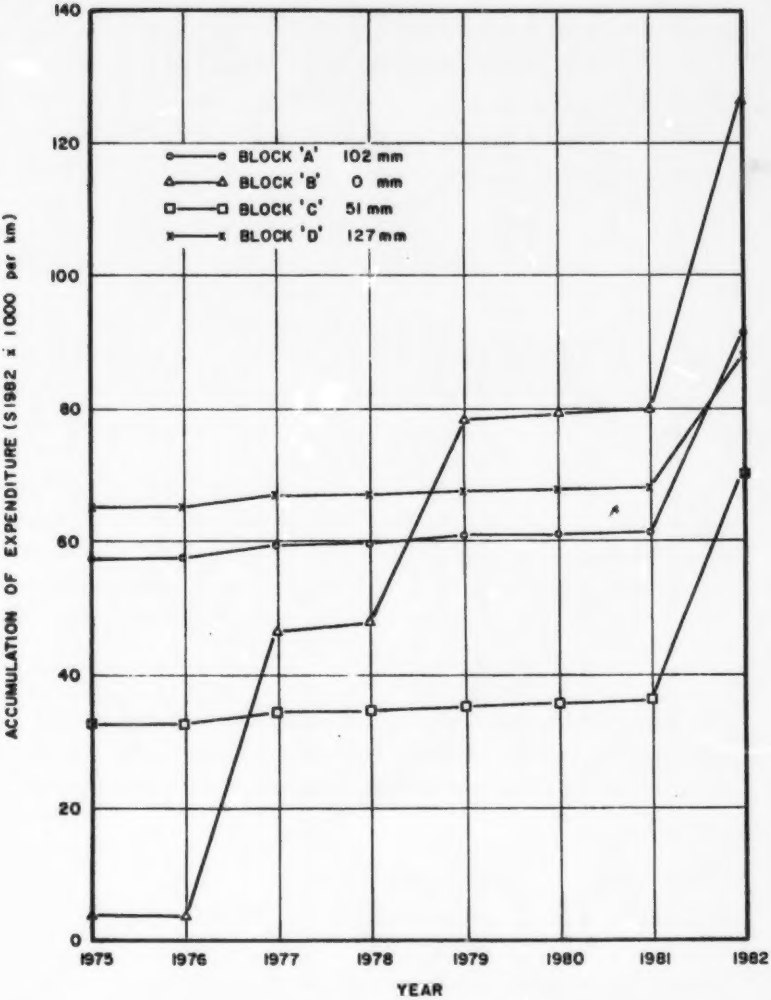

Figure 11/ Hot-Mix Price Index Present Worth, C.S. 1 - 10

TABLE 11

SAVINGS COMPARISON TO BLOCK D

YEAR	BLOCK 'D' - 'A'			BLOCK 'D' - 'A'			BLOCK 'D' - 'A'		
	EXPEND $/km	INTEREST $/km	BALANCE $/km	EXPEND $/km	INTEREST $/km	BALANCE $/km	EXPEND $/km	INTEREST $/km	BALANCE $/km
1975	3 925	-	3 925	29 654	-	29 654	16 300	-	16 300
1976	17	356	4 298	(45)	2 687	32 296	(45)	1 477	17 732
1977	(420)	378	4 256	(22 519)	2 868	12 645	-	1 583	19 315
1978	(183)	399	4 472	(469)	1 185	13 361	-	1 810	21 125
1979	(87)	459	4 844	(21 519)	1 371	(6 787)	-	2 167	23 292
1980	9	604	5 457	(402)	(846)	(8 035)	(216)	2 902	25 978
1981	(129)	717	6 045	(438)	(1 055)	(9 528)	-	3 411	29 389
1982	(10 063)	816	(3 202)	(27 077)	(1 286)	(37 891)	(14 589)	3 968	18 768
Additional Funded			22 925			57 614			(- 955)

NOTE: REHABILITATION ON BLOCK D IN 1982 COST $19,723

A. Performance Measurements

-Deflection

The deflection measurements show that thicker overlays reduce the deflection more than thinner overlays. There are indications that the base sandwich may not perform as well as the full depth asphalt overlay.

-Ride Comfort Index (RCI)

Thicker multi-lift overlays give higher initial Ride Comfort Index values than thin lift overlays.

Although the thicker overlays have had higher rates of RCI loss, they are still providing higher RCI values than the zero and 51 mm overlay sections. This implies a longer service life for thicker overlays.

-Crack Development

Cracks are reflecting through all overlaid sections. Prevention of reflective cracking requires more than the common overlay to treat a cracked pavement. Other methods have to be developed to prevent reflective cracking.

B. Maintenance Activities

The non-overlaid section shows that distressed pavements can be maintained cost effectively to retain the general ride. The pavement condition is improved by replacing cracked material with new material.

C. Economics

-Deep patching can be used to repair failures up to 5% of surface area.

-The choice of overlay thickness is a management trade-off of higher performance-higher cost versus lower performance-lower initial cost. This decision will be dependent on funds available and what alternative uses for the funds are required.

-Management has the choice of longer performance life for a few roads or a shorter performance life for many roads. This choice is going to vary from organization to organization.

- The trend of performance and cost analysis verifies many of the models found in many pavement management references.

References

1. The Asphalt Institute, "Asphalt in Pavement Maintenance," The Asphalt Institute, 1967, MS-16 Pg. 18-26.
2. RTAC Pavement Management Committee, Pavement Management Guide, RTAC, 1977, Pg. 3.9.
3. Saskatchewan Highways and Transportation, Surfacing Manual.
4. Province of Saskatchewan, Public Accounts 1978-79, Pg. B.8.

Session 4

Sealing Cracks in Flexible Pavements in Cold Areas

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SEALING CRACKS IN FLEXIBLE PAVEMENTS IN COLD AREAS

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1/ INTRODUCTION

Crack sealing has had a rather checkered (on-again, off-again) history in our Ministry (MTC). In many instances in the past, crack sealing was demonstrated to be ineffective due to both application procedures and materials available at the time.

The materials with which MTC has had such poor previous experience for crack sealing are the RS-type emulsions combined with sand or crushed stone chips. The method is to spray the RS emulsion over the crack with sand or stone chips spread manually over the emulsion (Figure 1). Usually, the result is ineffective because very little emulsion is able to penetrate the crack to effect a seal (Figure 2). Chance of success is not enhanced by the fact that crack sealing by this method is generally carried out in the summer when all the cracks are at their closed position (Figure 3).

However, the consequences of leaving cracks untreated to permit entry of water and brine from road salt can be severe. Premature deterioration, loss of functional serviceability, more extensive maintenance requirements, and decreased useful life are certain to occur.

In the summer, rainwater will enter the pavement structure through pavement cracks and tend to saturate the granular base and subbase. The excessive moisture will cause pumping under traffic (Figure 4) with resultant loss of structural integrity at, and immediately adjacent to, the crack. Net results can be development of multiple cracking or spalling (Figure 5).

During winter months, large quantities of brine solution from road salting are available for entry into the pavement structure through the crack which is now opened wider than in summer (Figure 6). Because of the lower freezing point of the saline solution which allows it to penetrate into the granular base, differential heaving usually results, causing the crack not only to open but to cup or lip as well (Figures 7 and 8). This cupping or lipping causes very rough ridability of the pavement surface in winter and a residue is quite common after the heave subsides during the spring and summer months. Even more drastic problems such as pot-holing (Figure 9) will occur if preventive maintenance action is not taken.

Well-timed preventive maintenance treatment is necessary to prevent all these problems. Crack sealing to waterproof the pavement is the logical solution regardless of past poor experience. Therefore, an effort is being made to search out more effective materials and better application techniques.

Three test sites were set up in 1980 and 1981 to look into performance of sealing materials in field conditions, and examine techniques for the preparation of the crack, rout design, pour methods, and equipment for sealing of pavement cracks by rout and seal. It is expected that the experiences gained at these and other sites will help to establish maintenance performance standards and materials specifications for crack sealing treatment.

This report describes recent Ontario experiences with the performance of different types of sealant, and discusses crack preparation, rout designs, crack filling methods and equipment for routing and sealing.

2/ THE PROBLEM

2.1/ Crack Movements

The mechanics of the opening and closing of cracks and the times during the year when these occur are more important to maintenance operations than determining why the pavement cracked.

From the year 1960 to 1964, MTC carried out an extensive investigation into flexible pavement cracking across the province [1]. Physical monitoring of the opening and closing of cracks in flexible pavements was done during this four-year period.

The investigation classified cracking into two main groupings according to their characteristics and behaviour patterns: the longitudinal cracks" and the "transverse cracks."

Results of the investigation showed the following.

1/ Longitudinal Cracks
- open, on the average, 3 to 4 mm during the winter months of January to March;
- close, on the average, 3 to 4 mm during the spring months of May to June; and
- generally have very small permanent residual openings.

2/ Transverse Cracks
- open, on the average, 7 to 10 mm during the winter months of January to March;
- close, on the average, 1.25 mm less than the winter opening during the spring months of May to June;
- generally suffer appreciable permanent opening which tends to increase with time;
- exhibit an annual rate of permanent residual crack opening of approximately 1.25 mm; and
- generally open twice as wide as the longitudinal cracks during the winter months.

2.2/ Traditional Materials and Treatments, and Their Performance

The materials which MTC traditionally used for crack sealing are the rapid-setting type emulsions with sand or stone chips for blotting purposes, and rout and seal with hot-poured or cold-poured rubberized asphalt sealants.

2.2.1/ Emulsion Crack Sealing

The performance standard for crack sealing with emulsion (RS-1) involves the preparation of the crack by brooming, followed by spraying of emulsion into and around crack; dry sand or stone chips are then thrown over the emulsion to blot any excess and prevent tracking by traffic (Figure 1).

Performance history of this treatment in Ontario is less than satisfactory. In the majority of cases it has been rated as totally ineffective (Figure 2). For the few 'successful' cases, the effective life has been very short.

The basic problem is crack movement. During the summer months, when this treatment is usually done, the cracks are at their closed position (Figure 3). Therefore, the chance of getting the emulsion to penetrate into the crack is quite remote. The result is merely surface dressing over the cracks. It is obvious, however, that even if the emulsion was able to penetrate into the crack, the winter opening (Figure 10) will render the emulsion ineffective because it simply does not have the capability to stretch and bridge this new gap.

There are also undesirable side-effects from this treatment: first, it lowers the aesthetic value of the highway (Figure 11); and secondly, and most importantly, it creates unwarranted roughness caused by the ridges of sand and emulsion around the treated cracks.

2.2.2/ Rout and Seal with Hot- or Cold-Poured Rubberized Asphalt Sealants

The performance standard for rout and seal involves the preparation of the crack by routing to a width and depth of 19 mm, followed by cleaning of the grooved crack by compressed air, and then pouring and filling flush to the pavement surface with Ministry-approved hot- or cold-poured rubberized asphalt sealants (Figure 12).

The performance history of this treatment in Ontario is mixed. In most cases it has been rated as marginal to acceptable with effective life up to three or four years (Figures 13 and 14). For the unacceptable cases, its effective life averages about one winter (Figure 15).

Again the basic problem is crack movement in winter. None of the approved sealants can maintain a perfect bond and will pull away from the pavement surfaces when the crack begins to open up during the winter months. Some sealants under low temperature became quite hard and, without the desirable flexibility, disbond or rupture from the cracks (Figures 16 and 17). This is so because sealants approved by MTC are basically specified for use as concrete pavement joint sealant. No differentiation is made between sealants used for flexible pavement crack sealing and for concrete pavement uses. In fact, the prime requisite for the sealant is the ability to seal against the intrusion of solids rather than flexibility. The performance history of sealants showed that all materials did seal against the entry of solids but because of disbonding or rupture could not prevent the penetration of liquids.

The other problems which bring about unacceptable performance include: poor preparation of the routed crack, such as dusty and oily contact surfaces of the grooved crack; poor application techniques, such as overheating or underheating of sealants; underfilling of the groove in the attempt to achieve flush fill as specified; and late cold weather or wet weather conditions during sealing.

Undesirable side-effects are also possible from this treatment. If the sealant disbonds or fails completely (Figure 15), the original crack is now changed into a uniform groove 19 mm wide and deep. This becomes, in effect, a perfect catchment for all surface water, especially during the critical winter months. Results can be drastic and quick deterioration such as spalling or break-up (Figure 18) will occur.

3/ REQUIREMENTS FOR IMPROVED PRODUCT

3.1/ Sealing Material Properties

Experience has indicated that sealants primarily designed for sealing concrete pavement joints are not entirely suitable for use in the sealing of flexible pavement cracks. Where the primary requirement of concrete pavement joint sealant is to seal against the entry of solids, the primary requirements of flexible pavement crack sealants are bond and flexibility under low temperature.

The desirable properties for flexible pavement crack sealant should be the following (in their order of importance):

1/ good adhesion between the sealant and the pavement surfaces year round,
2/ ability to retain same adhesion quality over time,
3/ flexibility at low temperatures and ability to bridge crack openings in winter without rupture or disbonding,
4/ flexibility at high temperatures and ability to withstand tracking from traffic, and
5/ flexibility in winter to withstand highway snow clearing operations without significant damage.

3.2/ Application Methods

The present standard of 19 mm wide and deep routed groove is derived from the desire to meet the shape factor which is an important consideration for concrete pavement joint sealant. Since MTC does not differentiate between sealing of concrete pavement and flexible pavement, this same standard is used for both maintenance activities.

However, in the matter of sealing flexible pavement cracks, the shape factor of the sealant is not important. The important factors are the ability to remain bonded and sufficient flexibility to bridge crack openings in the winter without rupturing.

The desired effects of the application methods are, therefore, as follows.

1/ Provide a clean bonding surface for the sealant to effect a permanent bond between the pavement surfaces and the sealant.
2/ Provide sufficient bonding surface for the sealant with minimal stress during crack opening in the winter to prevent disbonding.
3/ Provide sufficient sealant so that when stretched due to crack opening in the winter, there will still be enough material thickness to bridge over the opening without rupturing.

3.2.1/ Crack Preparation

There are two ways to prepare the crack for sealing. One is routing the crack to provide a clean and uniform bonding surface and a reservoir for the sealant to compensate for the expected elongation due to crack opening in the winter. The other is applying the sealant directly over the crack like a band-aid, that is, no special preparation except cleaning.

3.2.1.1/ Routed Crack

Crack routing is expensive. It is labour-intensive with low productivity, high energy consumption, and high materials (router bits) cost. The larger the rout size, the more it costs.

Since the routed crack is designed as a reservoir for the sealant, a smaller rout size may perform equally well compared against the standard. The Ministry has now tried 16 mm and 10 mm rout sizes.

3.2.1.2/ Band-Aid

Large savings in cost and time will be achieved if sealant is applied directly over the crack like a band-aid, without routing. This method, too, has been tried.

3.2.1.3/ Cleaning

Cleaning of dust and oil from the crack before sealant application is very important. Proper cleaning of the crack is a must because it contributes directly to achieving proper bonding of the sealant to the pavement surface.

Crack routing will create a film of fine dust which clings to the pavement surface and in the routed crack. This dust must be removed to effect proper bonding.

3.2.2/ Sealant Application

The standard application specified is that the sealant must be poured flush with the pavement surfaces. However, past experiences showed that if the sealant is applied so that it overlaps slightly on each side, the sealant has a higher survival rate for disbonding.

Various applications have been tried:

1/ overlap - band width 50 mm

2/ overfill - band width 80 mm with or without routing

3/ overfill and strike off - overfill to 80 mm and squeegee

4/ overfill and shaped - overfill to 80 mm and shaped with a converted squeegee to form a bead over the crack

See Figures 19, 20, 21, 22, 23.

3.3/ Equipment Problems

Crack sealing is a labour-intensive activity assisted by a number of small mechanical equipment and hand tools.

3.3.1/ Routing Equipment

There are three mechanical routers available and commonly used in Ontario.

1/ The Windsor, a vertical router with carbide cutter, has a very low production rate and accomplishes, on the average, only 0.72 m/min, but it has the ability to follow the meandering of the crack because of its light weight and manoeuvrability (Figure 24).

2/ The Crafco, a rotary router with toothed steel cutting wheels, has a much higher production rate at 8.1 m/min. It has good manoeuvrability in following the meandering but the Windsor Router is still required at areas of very sharp twists and turns (Figure 25).

3/ The Tennant, a rotary-striking router with carbide-tipped hammers, has a very high production rate of 15.2 m/min. This equipment is designed primarily for routing and cleaning concrete pavement joints. It is fast and efficient but it is a straight line performer and incapable of following the slightest twist or turn (Figure 26).

3.3.2/ Cleaning Equipment

There are two commonly-used types of cleaning equipment in Ontario plus one recently-introduced unit.

1/ Compressed Air - is a standard compressor which produces the necessary high pressure air-jet for removing dirt and dust from routing of the crack. The recommended air-jet pressure is 630 kPa (90 psi) minimum. The chief disadvantage of this unit is its bulk and restricted movement because it must be moved by towing with a truck (Figure 27).

2/ Back-Pack Blower - is operated by a small engine capable of producing an air velocity of 320 km/h, maximum. However, experience indicates that unless the blower is in top mechanical condition, the unit does not perform adequately. The advantage is its complete mobility and freedom of action (Figure 28).

3/ Hot Compressed Air - a standard air compressor is coupled to a propane supply unit and both gases are metered together as a mixture and ignited in a special chamber before release. The result is a superheated air-jet. The maximum temperature of air-jet delivered at 700 kPa (100 psi) is 1100°C. It blows clean as well as dries out any moisture in the routed groove and in the crack to achieve better bonding for the sealants. Performance under operational conditions for this unit (trade name Kardey (HCA) Lonce) is still unknown but it is now being evaluated in the field by MTC (Figure 29).

3.3.3/ Application Equipment

There are a number of manufacturers who make kettles for the hot-pour sealing materials. They are all double-jacket types with heat transfer through the medium of oil heated by propane burners. They are all equipped with agitators, either paddles or augers (Figure 30).

Once the sealants are ready for pouring, there are a number of applicators available which range from primitive to sophisticated.

The primitive include cone-applicator, garden watering can or facsimiles. They all must be heated by direct heat to remove build-up from cooling. The cone-applicator can be more sophisticated because it has a hand-controlled stopper and a seat in the kettle for heating to remove build-up and maintain free-flow (Figure 31). On the whole, quality depends on the operator's care and proper supervision.

The heat-wand applicator is part of the kettle, and sealant is pumped out through a heat-resistant, flexible rubber-metal hose and poured through a metal nozzle. When not in use, the nozzle must be placed inside the kettle and material kept circulating by pump to prevent jelling inside the hose and nozzle. Rate of application is controlled by a valve in the nozzle (Figure 30). Our maintenance staff finds this applicator to be efficient and tidy. Production rate is high with a high degree of control. Performance rating is very satisfactory.

3.4/ Laboratory Testing

The Ministry accepts or rejects a sealant based on specification MTC Form 1212 (Figure 32). This specification covers hot-poured rubberized asphalt joint sealing compounds for sealing joints in Portland cement or asphaltic concrete with a small amount of movement. The specification conforms to the requirements of ASTM Designation: D1190(74) Concrete Joint Sealer, Hot-Poured Elastic Type. The only modification is that all three bond test specimens shall develop no crack or separation in the sealing compound or between the sealing compound and sawn Portland cement or asphaltic concrete block.

The Ministry does not differentiate between asphalt and concrete type sealers on the designated source list acceptance test.

Table 1 shows the sealants listed in the MTC Designated Sources list.

4/ EXPERIMENTS

Three areas, located on Highway 16 and 17 of the Ottawa District, and on Highway 69 of the Huntsville District, have been specifically designed to address the questions of:

1/ performance of sealants which are on the Ministry-approved list,
2/ performance of sealants which are not on the Ministry-approved list,
3/ performance of different rout and seal configurations and methods, and
4/ cost and performance evaluation of rout and seal as a full-scale maintenance operation.

4.1/ Full-Scale Maintenance Operation - Ottawa District

The experiment in Ottawa addresses the following:

1/ cost of rout and seal treatment as an in-house operation,
2/ comparison of in-house cost vs. privatization,
3/ quality of work comparison between in-house and private sector,
4/ performance of different sealants over time,
5/ performance of different rout width and depth, and no routing (Band-Aid),
6/ performance of different application methods for sealant,
7/ effective life of sealants over time, and
8/ cost-effectiveness of treatment and consequences of no rout and seal treatment.

The test areas were first mapped with a crack survey before the rout and seal treatment was done.

A special work-crew was formed and trained to perform this specific operation. The equipment they used was standardized with a Crafco router, air compressor for cleaning, and a Crafco kettle equipped with a heat-wand applicator for pouring.

Monitoring is carried out in the winter months with physical measurements and visual evaluation to obtain the following information:

1/ performance of various sealants used,
2/ performance of different rout and seal configurations and methods,
3/ mapping of new cracks developed after the rout and seal treatment,
4/ effectiveness of various sealants used, and

5/ rate of deterioration of overall pavement performance for sealed sections against the unsealed control section.

Performance is monitored by physically measuring the percentage of sealant disbonding in low temperatures. The evaluation standard is:

1/ single bond failure - disbonding of sealant on one side of the routed and sealed crack, or single rupture of the sealant; and

2/ double bond failure - disbonding of sealant on both sides of the routed and sealed crack, or multiple rupture of the sealant.

The total length of bond failure measured is divided by the total length of sealed crack to obtain the percentage. Other evaluations are made visually for resiliency in low temperatures, plow damages, and effectiveness in keeping brine solution out.

The sealants tested were Ministry-approved Hydrotech 6160, Meadows 164R, and Hi-Spec, plus a non-approved sealant, Shell Cariphalte.

The rout and seal configurations and methods tested were 19 mm and 10 mm overlap (50 mm), 19 mm overfill (80 mm), 19 mm overfill and strike-off (80 mm plus), 19 mm overfill and shaped (80 mm plus), and band-aid overfill (80 mm). (See Figures 19 to 23 inclusive.) Table 2 summarizes the information for the Ottawa test to date.

4.2/ Comparative Field Testing - Huntsville District

The experiment in Huntsville addresses the following:

1/ performance of sealants which are on the Ministry-approved list, and

2/ performance of sealants which are not on the Ministry-approved list.

Manufacturers of sealants were invited to participate in this comparative field test. The Ministry provided the site on Highway 69 north of

Port Severn plus partial compensation at a fixed rate per metre of crack sealed. The participating members placed their products to their own specifications and under their own supervision.

The first comparative field test was held in summer, 1980. All sealants were placed in 19 mm rout and filled flush, with four exceptions which were overfill and strike-off (80 mm band width). Six sealant manufacturers participated.

The second comparative field test was called in 1981 and a total of nine manufacturers participated, four of them held over from 1980. All sealants were placed in 19 mm rout and overfill with strike-off.

Details of this experiment were reported in 1981 and 1982 [2,3]. Performance is monitored in the winter by visual observation and Table 3 summarizes the information for this Huntsville test to date.

5/ OBSERVATIONS AND DISCUSSIONS

5.1/ Performance of Field Tests

Assessments were made in late winter when conditions are generally at their worst, during 1980/81 and 1981/82.

5.1.1/ Full-Scale Maintenance Operations - Ottawa District

Observations of Highway 16 test site (1980-82) included the following.

1/ The Ministry approved Hydrotech 6160 and Meadows Hi-Spec applied to 19 mm rout and overfill (80 mm band width) performed satisfactorily in its first winter.

2/ The non-approved Shell Cariphalte applied to 19 mm rout and no routing, and overfill (80 mm band width) performed exceptionally well in its first winter.

3/ In the second winter, both Hydrotech 6160 and Meadows Hi-Spec had more than 50% bond failures.

4/ In the second winter, Shell Cariphalte applied to rout and overfill still performed satisfactorily.

5/ In the second winter, Shell Cariphalte applied as band-aid without routing showed more than 70% bond failure.

6/ Snowplow damage of Shell Cariphate applied as band-aid was reported in first winter. Damage in the form of shaving-off by the plow blade was reported as extensive by the snow clearing crew.

7/ No plow damage was reported from rout and seal cracks.

8/ All three sealants effectively resealed over the cracks during the summer months.

Observations of Highway 17 test site (1981-82) included the following.

1/ The approved Hydrotech 6160 applied to 19 mm and 10 mm rout, and overlap (50 mm band width) performed unsatisfactorily with more than 50% bond failures the first winter.

2/ The same Hydrotech 6160 applied to 19 mm rout and overfill with strike-off performed unsatisfactorily the first winter with more than 60% bond failures.

3/ The same Hydrotech 6160 applied to 19 mm rout and shaped to form a bead over the routed crack (overfill and shaped) performed satisfactorily the first winter.

4/ The approved Meadows 164R does not perform at all well, with a failure rate that ranged from 50% to 96% the first winter.

5/ The non-approved Shell Cariphalte applied to 19 mm and 10 mm rout and overlap (50 mm band width) performed satisfactorily the first winter.

6/ Although bond failures range up to 96%, sealants all remained intact in routed cracks.

7/ The Shell Cariphalte displayed the most flexibility under low temperatures; Hydrotech 6160 was still somewhat resilient, but Meadows 164R and Hi-Spec were rigid.

8/ The Shell Cariphalte recovered best under high temperatures during the late spring and summer.

9/ All four sealants effectively resealed the routed cracks during the spring months.

10/ More new cracks developed on the control section (no sealed cracks) than all sealed sections during the winter months.

5.1.2/ Comparative Field Testing - Huntsville District

Observations of Highway 69 test site (1980) included the following.

1/ The four cold-poured sealants failed either immediately or in the first winter and were all removed from the test in 1981. Failures were complete disbonding and actual loss.

2/ The approved Hydrotech 6160 and the non-approved 6165 performed satisfactorily the first winter.

3/ The non-approved Be-Ram Supergook performed satisfactorily the first winter.

4/ All other sealants including the approved Meadows 164R performed from marginally acceptable to unacceptable the first winter.

5/ In the second winter only the approved Hydrotech 6160 and the non-approved Be-Ram Supergook performed satisfactorily.

6/ All other sealants performed either marginally or unsatisfactorily in the second winter.

7/ Minor loss of sealants from cracks occurred on all materials performing satisfactorily.

8/ All the hot-poured sealants effectively resealed the routed cracks during the summer months.

Observations of Highway 69 test site (1981) included the following.

1/ The two cold-poured sealants failed in the first winter.

2/ The non-approved Shell Cariphalte performed satisfactorily the first winter.

3/ The approved Hydrotech 6160 and the non-approved 6165 showed acceptable performance the first winter.

4/ The performance of the approved Meadows Hi-Spec and the 164R was marginally acceptable the first winter.

5/ All other hot-poured sealants performed unsatisfactorily.

5.2/ Discussion of Results

5.2.1/ Materials

Sealant for flexible pavement cracks must have three important qualities: the ability to bond itself to the pavement surface, the ability to remain flexible without disbonding or rupturing under low temperature conditions, and the ability to maintain the former two over time.

The cold-poured sealants which underwent testing simply did not meet these three requirements. All failed completely either at the time of application or shortly after.

The hot-poured sealants also showed disappointing performances. All appeared satisfactory until the first winter. As temperature decreased, most sealants became less and less flexible. At the same time, the crack began to open up due to the low temperatures. This combined hardening of the sealant and crack opening put such a stress at the interfaces of the sealant and the grooved crack surfaces that the sealant became disbonded and/or ruptured (Figures 16 and 17).

The exceptions are the Shell Cariphalte and the Be-Ram Supergook. These two sealants performed with acceptable bonding and flexibility these past two winters. How long lasting this performance will be, however, remains unknown because both materials are recent arrivals to the marketplace. But based on their performances to-date, a safe estimate will be at least two more winters of satisfactory performance.

For the traditional sealants listed on the Ministry-approved designated sources, only the Hydrotech 6160 and the Meadows Hi-Spec remained reasonably flexible during the winter to maintain acceptable bonding performance after two winters. But based on their performances to-date, durability will be approximately three years, with maximum four years of life expectancy.

5.2.2/ Application Procedures

Routing is done to form a uniform groove for achieving the required shape factor so desirable for concrete pavement joint sealing. For flexible pavement, routing is intended to create a reservoir of sealant to accommodate any elongation of the sealant because of winter crack opening. The standard 19 mm rout size is designed to accommodate both requirements.

But this standard showed no appreciable difference in performance from the smaller rout sizes of 16 mm and 10 mm. The reason is that when the sealants harden under low temperatures and lose their flexibility, the amount of sealant in the grooved crack is no longer a factor to accommodate any elongation which will prevent disbonding or rupturing.

There were no apparent differences in the performance of the application of overlap (50 mm), overfill (80 mm), and overfill with strike-off/methods. The rout also made no difference in performance. Again, if the sealants hardened and lost their flexibility, overlap, overfill, or overfill with strike-off does not prevent disbonding or rupturing.

The overfill and shaped with bead application method again demonstrated it will help only when the sealant is adequate.

The band-aid without routing method is hard to justify because of snow clearing damage and the thin membrane which will rupture from over-stretching due to crack opening.

5.2.3/ Equipment

Equipment improvement or new equipment designed for flexible pavement rout and seal operation is definitely required.

Again, the basic problem with the routing equipment is that it is primarily designed for routing concrete pavement joints, and forming the all important shape factor for the sealant. Experience indicates that only the most unproductive router is capable of following the meandering cracks generally occurring on the flexible pavements. The more efficient the router, the less manoeuvrable it is because of its weight and bulk.

Cleaning of the routed crack before sealing also needs new or more efficient equipment. Experience indicates that the compressed air and the back-pack blower cannot entirely overcome the problem of coating of the routed crack by a film of fine dust. If the pavement is moist or if the operation is carried out immediately after rain, a fine slurry deposit will develop in the routed crack and the present equipment just cannot overcome this problem and clean off the slurry. Experience also showed that unless dust or slurry are removed, the sealant will not bond properly and the chance of sealant failure is near 100%.

A common problem encountered on the application is that all kettles lack a proper temperature indicator to measure accurately the sealant melting temperature. Because of this shortcoming, underheating or overheating of the sealant occurs rather frequently.

5.3/ Problems Still to be Resolved

More and more maintenance funds are being spent on rout and seal operations. Therefore, problems which exist in this operation must be resolved soon.

5.3.1/ Application Techniques and Equipment

A more appropriate routing configuration for flexible pavement should be devised. Sufficient material to accommodate stress and elongation due to crack movement are the main factors to be considered, not the sealant shape factor.

More effective rout and seal equipment for flexible pavement should be developed through encouragement to the private sector. Equipment such as Hot-Compressed Air lance (HCA) is a prime example (Figure 29).

5.3.2/ Materials

Only two out of all these sealants tested are considered as satisfactory. Therefore, there is an urgent need to bring the proper materials on-stream. The adequacy of materials is of the utmost importance because if the materials fail, the best application techniques and equipment are of no avail.

5.3.3/ Changes In Specification

The two sealants performing satisfactorily do not pass our MTC specification tests. Materials on the approved list do not perform or are only marginally acceptable. This indicates that the specifications for sealant of concrete pavement joint do not necessarily meet the requirements for sealing flexible pavement cracks. Therefore, this question should be resolved immediately.

6/ RECOMMENDATIONS

It was observed that the unsealed control section had developed more new cracks than the sealed sections, whether the sealant performed satisfactorily or not. This indicates that although the sealant may not perform as perfectly as we wish, the fact the sealant stays in the routed crack means it is still effective to some degree in performing its intended function, that is, keeping water out.

Therefore, action is needed to improve the present maintenance standards and materials specifications by the following.

1/ Review the present standard rout configuration of 19 x 19 mm for appropriateness.

2/ Adopt a shallower rout, say 10 x 10 mm, while the search for the most effective and safe rout and seal configuration, equipment, and method is accelerated.

3/ Review the present laboratory acceptance test for sealants of flexible pavement. A radically new set of tests and acceptance criteria may need to be developed.

4/ Review the present Ministry-approved list to note sealant materials which are unacceptable if used on flexible pavement.

5/ Develop a new approval list for flexible pavement sealant.

6/ As a temporary measure, promote the use of the two non-approved sealants with satisfactory field performance history, namely, the Shell Cariphalte and Be-Ram Supergook.

7/ REFERENCES

[1] R. Ian Kingham, "Cracking of Asphalt Concrete Pavements - A Pilot Study Parts I, II & III," February 1965, Ministry of Transportation and Communications, Ontario.

[2] R.C. Evers, "Evaluation of Crack-Sealing Compounds for Asphaltic Pavements," EMO Report 27, December 1980.

[3] R.C. Evers, "Evaluation of Crack-Sealing Compounds for Asphaltic Pavements," EMO Report 27A, December 1981.

Table 1/ DESIGNATED SOURCES FOR JOINT SEALING COMPOUNDS (Hot-Poured/Cold-Poured Rubberized Asphalt)

Supplier	Product Name	Pouring Temperature °C
BAKELITE Inc.	FLINTSEAL 590-13	195-215
W.R. Meadows of Canada Ltd.	(a) Sealtight 164R	195-215
	(b) Hi-Spec	195-215
HYDROTECH Membrane Corp	Sealz 6160	195-215
FLINTKOTE Company of CANADA Ltd.	Flintseal 570-05	*Applied COLD
W.R. Meadows of Canada Ltd.	No. 158	*Applied COLD

*Cold-Poured Rubberized Asphalt accepts or rejects based on ASTM Specification(s) D 1850.

Table 2/ EVALUATION OF CRACK SEALING COMPOUNDS AND ROUT AND SEAL METHODS FOR ASPHALT PAVEMENTS

HIGHWAY 16/ KEMPTVILLE ROAD TO HIGHWAY 43

HIGHWAY 17/ HIGHWAY 44 WESTERLY

HWY	SECT NO.	MANUFACTURER	POUR METHOD (HOT)	CRACK ROUTED			TOTAL CRACK LENGTH SEALED (M±)	DATE LAID	PERFORMANCE EVALUATION					
									81-02-24			82-03-10		
									% BOND FAILURE			% BOND FAILURE		
				YES	NO	SIZE (mm)			SINGLE	DOUBLE	TOTAL	SINGLE	DOUBLE	TOTAL
16	1	HYDROTECH 6160	OVERFILL	✓		19 x 19	428	80-07-22	-	-	3.0	-	-	50.5
	2	MEADOWS HI-SPEC	OVERFILL	✓		19 x 19	443	80-07-22	-	-	16.7	-	-	66.0
	3	SHELL CARIPHALTE	OVERFILL	✓		19 x 19	425	80-07-22	-	-	0	-	-	23.7
	4	SHELL CARIPHALTE	OVERFILL		✓		382	80-07-22	-	-	0.6	-	-	73.5
17	1	HYDROTECH 6160	OVERLAP	✓		19 x 19	504	81-06-23				11.0	47.4	58.4
	2	CONTROL SECTION												
	3	SHELL CARIPHALTE	OVERLAP	✓		19 x 19	475	81-06-23				0.6	16.9	17.5
	4	SHELL CARIPHALTE	OVERLAP	✓		10 x 10	562	81-06-23				4.1	19.6	23.7
	5	HYDROTECH 6160	OVERLAP	✓		10 x 10	1102	81-06-23				12.2	45.1	57.3
	6	MEADOWS 164R	OVERLAP	✓		10 x 10	331	81-06-23				0.2	95.6	95.8
	7	MEADOWS 164R	OVERLAP	✓		19 x 19	353	81-06-23				1.7	89.5	91.2
	8A	MEADOWS 164R	OVERFILL STRIKE OFF	✓		19 x 19	166	81-06-23				-	-	96.3
	8B	MEADOWS 164R	OVERFILL SHAPED	✓		19 x 19	108	81-06-23				-	-	54.0
	8C	HYDROTECH 6160	OVERFILL SHAPED	✓		19 x 19	155	81-06-23				-	-	20.3
	8D	HYDROTECH 6160	OVERFILL STRIKE OFF	✓		19 x 19	184	81-06-23				-	-	67.1

Table 3/ EVALUATION OF CRACK SEALING COMPOUNDS FOR ASPHALT PAVEMENTS

HIGHWAY 69/ PORT SEVERN NORTH NORTHLY

SECT NO.	MANUFACTURER	TYPE		ROUT (mm)	POUR METHOD	DATE LAID	PERFORMANCE EVALUATION			
							81-01-28		82-03-02	
		HOT	COLD				APPEARANCE	BOND	APPEARANCE	BOND
7	HYDROTECH 6160	✓		19 x 19	FLUSH	80-07-07	EXCELLENT	EXCELLENT	GOOD	ACCEPTABLE
8	BE-RAM SUPERGOOK	✓		19 x 19	OVERFILL STRIKE OFF	80-07-07	GOOD	EXCELLENT	GOOD	ACCEPTABLE
9	HYDROTECH 6165	✓		19 x 19	FLUSH	80-07-08	EXCELLENT	EXCELLENT	FAIR	BORDERLINE
10	FLINTKOTE X-8001		✓	16 x 16	FLUSH	80-07-09	FAILED ON	DAY OF POUR SECTION		REMOVED
11	CHEVRON CRACKFULL		✓	19 x 19	FLUSH	80-07-14	FAILED	FAILED	SECTION REMOVED	
12	MEADOWS 164R	✓		19 x 19	FLUSH	80-07-16	GOOD	GOOD	POOR	UNACCEPTABLE
13	FLINTKOTE X-8006		✓	19 x 19	FLUSH	80-07-16	POOR	POOR	FAILED SECTION REMOVED	
14	MEADOWS 164	✓		19 x 19 16 x 16	FLUSH	80-07-16	FAIR	FAIR-GOOD	POOR	UNACCEPTABLE
15	FLINTKOTE X-8013	✓		19 x 19	FLUSH	80-07-17	FAIR	FAIR	POOR	FAILED
16	MEADOWS SOF-SEAL	✓		19 x 19	FLUSH	80-07-17	GOOD	FAIR	POOR	UNACCEPTABLE
17	CRAFCO OVER-FLEX MS (MOD)	✓		19 x 19	OVERFILL STRIKE-OFF	80-07-22	POOR	POOR	POOR	FAILED
18	CRAFCO OVERFLEX MS	✓		19 x 19	OVERFILL STRIKE-OFF	80-07-22	FAIR	FAIR	POOR	FAILED
19	FLINTKOTE X-8033		✓	19 x 19	FLUSH	80-27-23	VERY POOR	VERY POOR	FAILED REMOVED	
20	BAKELITE FLINTSEAL 590-13	✓		19 x 19	OVERFILL STRIKE-OFF	81-07-06			FAIR	UNACCEPTABLE
21	HYDROTECH 6165	✓		19 x 19	SAME	81-07-07			GOOD	ACCEPTABLE
22	CRAFCO ROAD-SAVER 201	✓		19 x 19	SAME	81-07-07			FAIR	UNACCEPTALBE
23	HYDROTECH 6160	✓		19 x 19	SAME	81-07-07			GOOD	ACCEPTABLE
24	PARASEAL 2065	✓		19 x 19	SAME	81-07-08			POOR	UNACCEPTABLE
25	PARASEAL 2070	✓		19 x 19	SAME	81-07-08			POOR	FAILED
26	MEADOWS 164R	✓		19 x 19	SAME	81-07-13			FAIR	ACCEPTABLE TO BORDERLINE
27	SHELL CARIPHALTE	✓		19 x 19	SAME	81-07-13			GOOD	GOOD
28	MEADOWS SOF-SEAL	✓		19 x 19	SAME	81-07-13			POOR	FAILED
29	MEADOWS HI-SPEC	✓		19 x 19	SAME	82-07-14			FAIR	ACCEPTABLE TO BORDERLINE
30	CHEVRON CIM			19 x 19	SAME	81-07-14			POOR	UNACCEPTABLE
31	SWEPCO COLD POUR		✓	19 x 19	SAME	81-07-15			POOR	FAILED
32	IBIS PRIS-MUSEAL A-1			19 x 19	SAME	81-08-24			POOR	FAILED

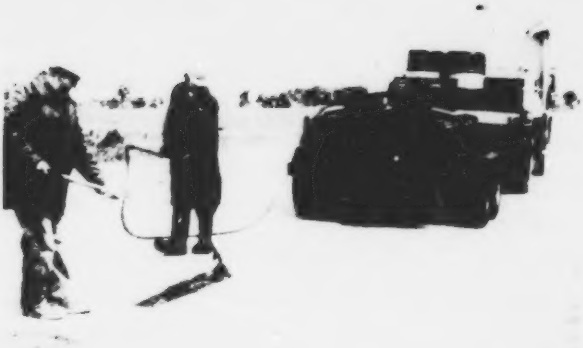


Figure 1/ Emulsion Crack Sealing Method

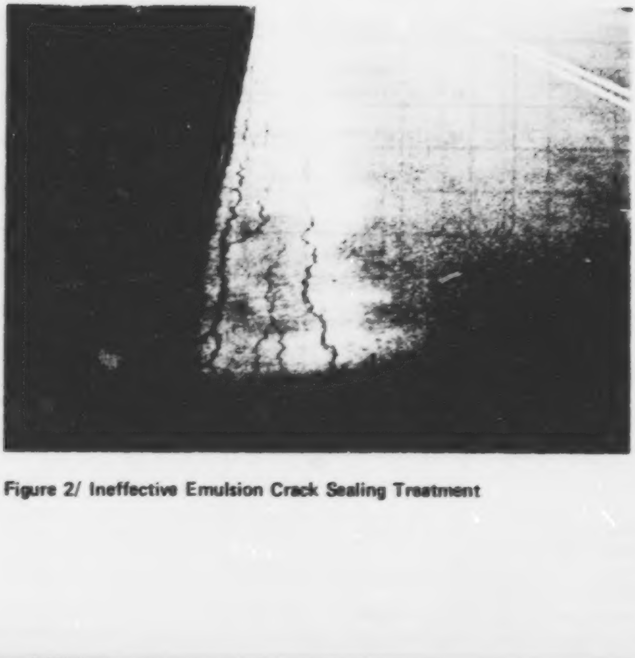


Figure 2/ Ineffective Emulsion Crack Sealing Treatment

Figure 3/ Crack at Closed Position in Summer Months

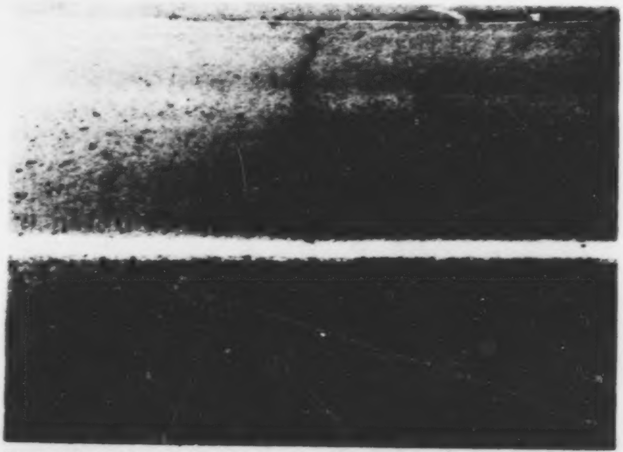


Figure 4/ Pumping Under Traffic Due to Excess Moisture
Entering Pavement Through Crack

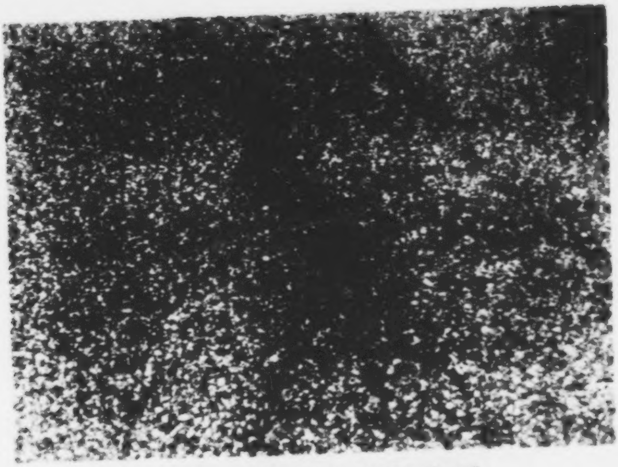


Figure 5/ Crack Spalling and Multiple Cracks Developed from Open Crack

Figure 6/ Crack Open Wide in Winter Allowing Brine Solution from Road Salting to Enter Pavement

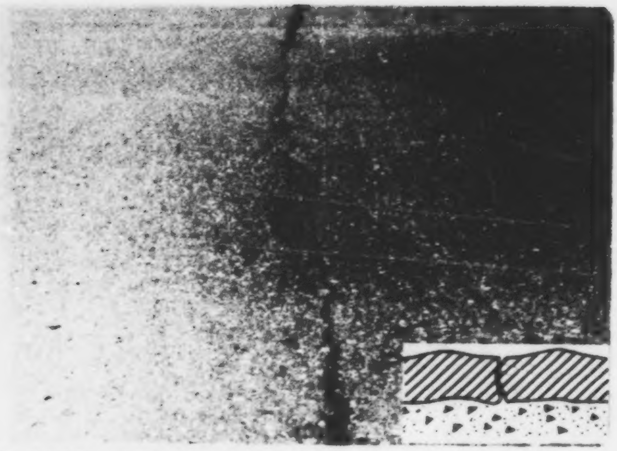


Figure 7/ Cupping of Transverse Crack

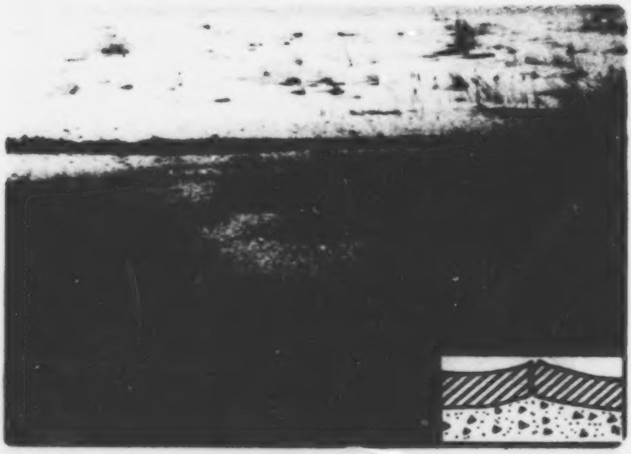


Figure 8/ Lipping of Transverse Crack

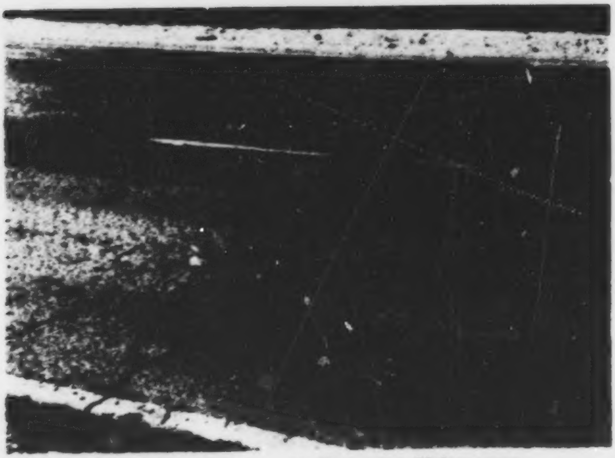


Figure 9/ Pot-Hole Resulted from No Preventive Maintenance

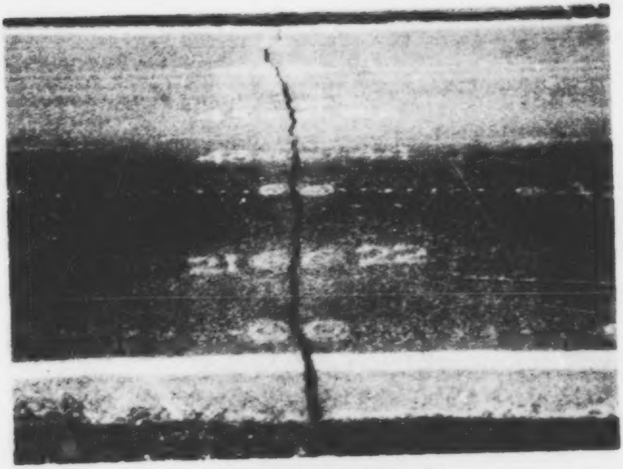


Figure 10/ Winter Opening of Crack from Figure 3

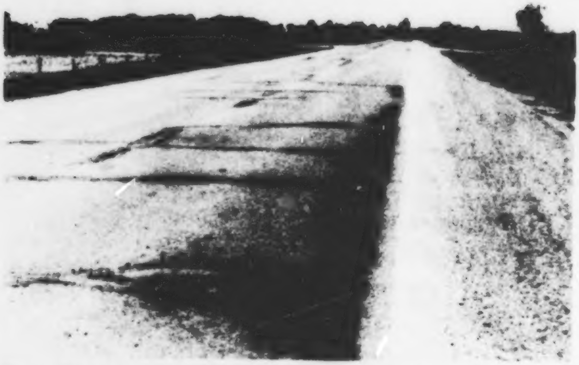


Figure 11/ Unsightly View of Pavement from Emulsion Crack Sealing Treatment

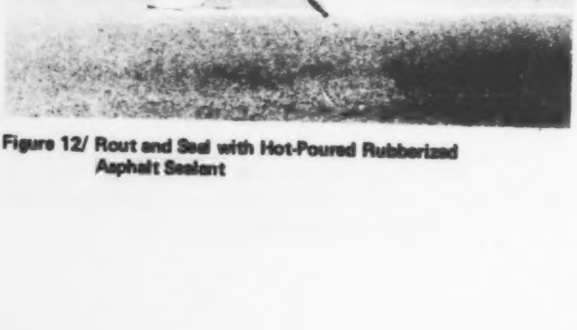


Figure 12/ Rout and Seal with Hot-Poured Rubberized Asphalt Sealant

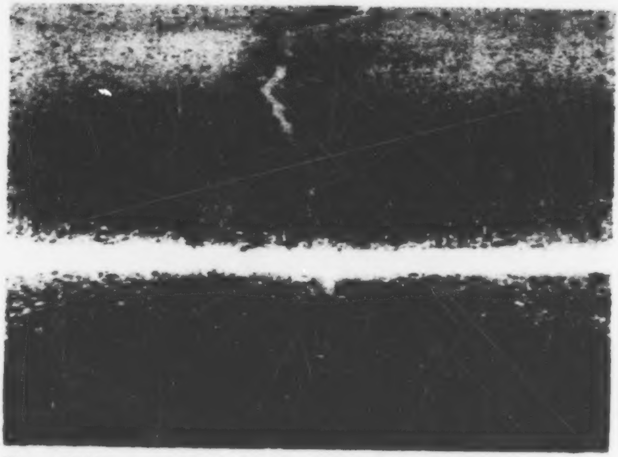


Figure 13/ Rout and Seal in Summer 1977, Age: 4 Years

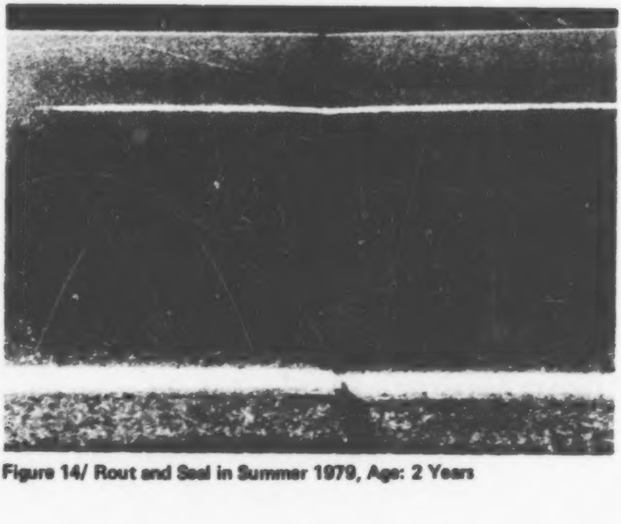


Figure 14/ Rout and Seal in Summer 1979, Age: 2 Years

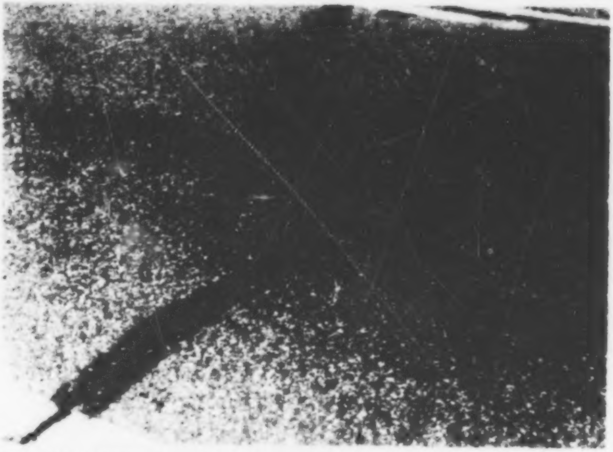


Figure 15/ Rout and Seal in Summer 1980, Age: 1 Winter

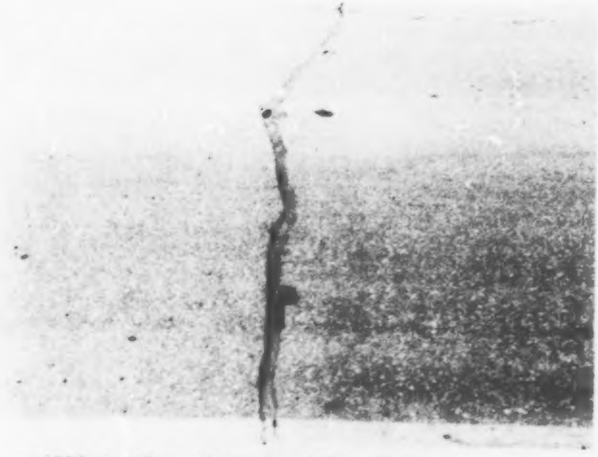


Figure 16/ Sealant Disbond from Routed Crack in Winter
Due to Hardening Under Low Temperatures

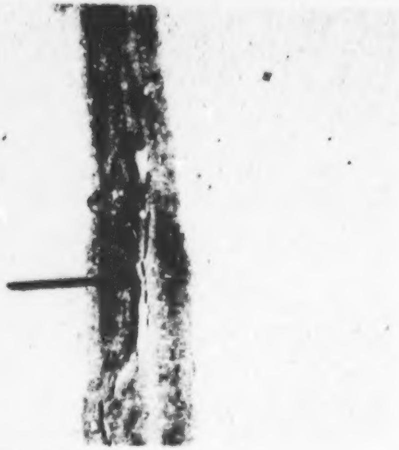


Figure 17/ Sealant Rupture in Winter Due to Hardening Under Low Temperatures

Figure 18/ Breakup of Pavement from Failed Rout and Seal Treatment

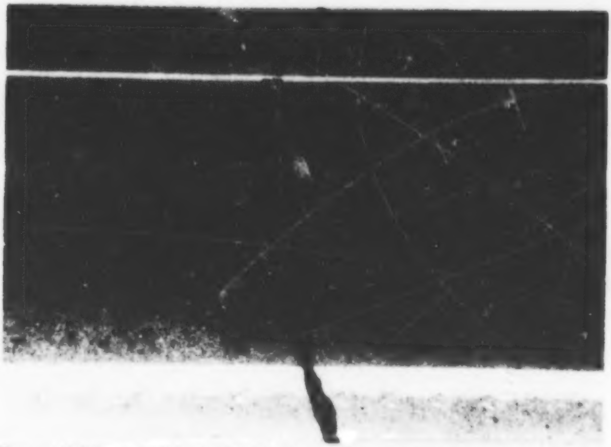


Figure 19/ Rout and Seal with Overlap (50 mm Band)

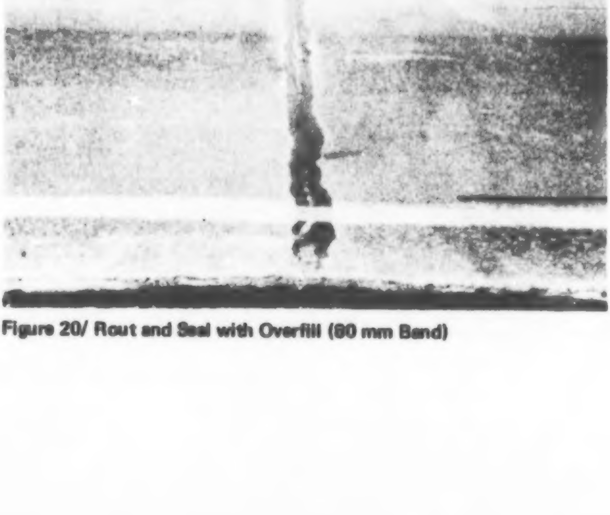


Figure 20/ Rout and Seal with Overfill (80 mm Band)

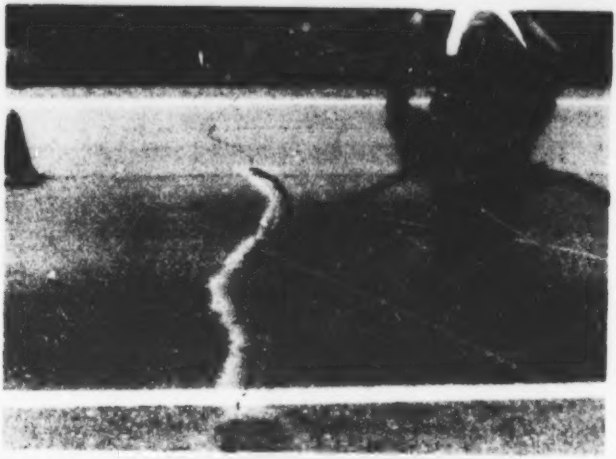


Figure 21/ Band-Aid Seal without Routing, Overfill (80 mm Band)

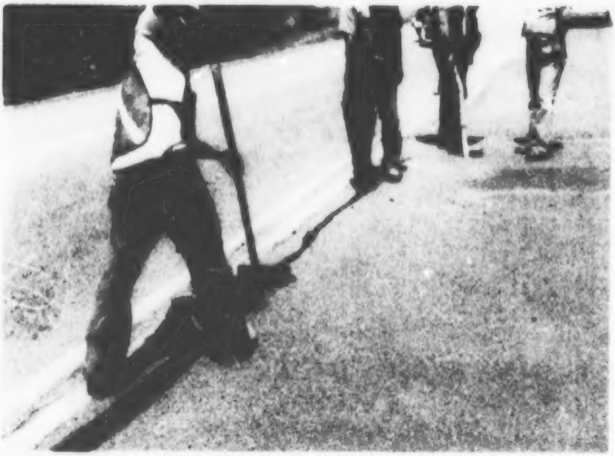


Figure 22/ Rout and Seal with Overfill-Strike Off (80 mm Band)

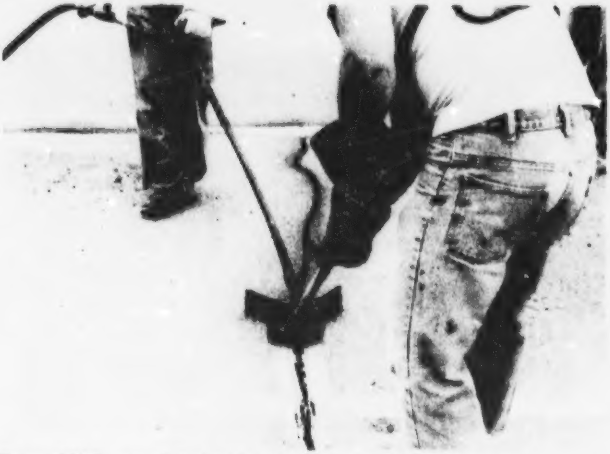


Figure 23/ Rout and Seal with Overfill-Shaped (80 mm Band)

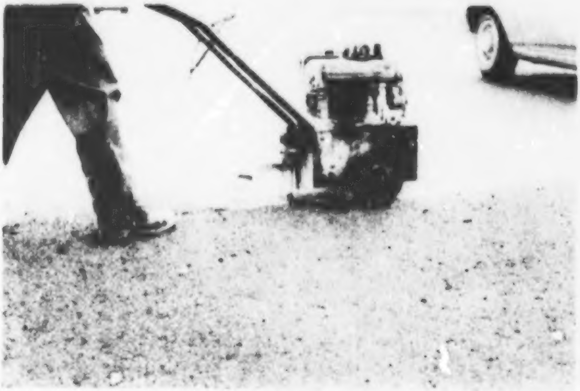


Figure 24/ Vertical Router (Windsor)

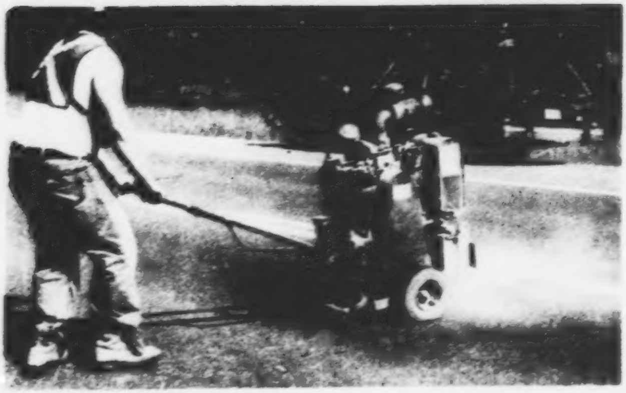


Figure 25/ Crafco Rotary Router

Figure 26/ Tennant Rotary-Striking Router

Figure 27/ Air Compressor

Figure 28/ Backpack Blower

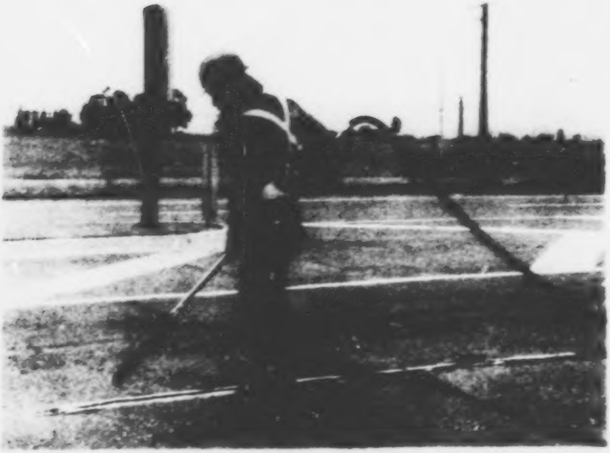


Figure 29/ Hot Compressed Air Lance

Figure 30/ Rubber Asphalt Sealant Kettle (Crafco)
with Heat Wand Applicator

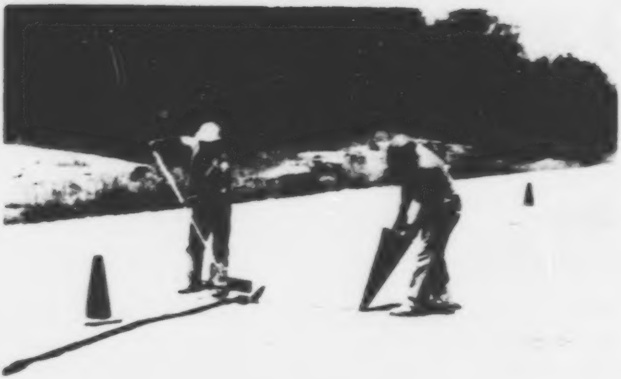
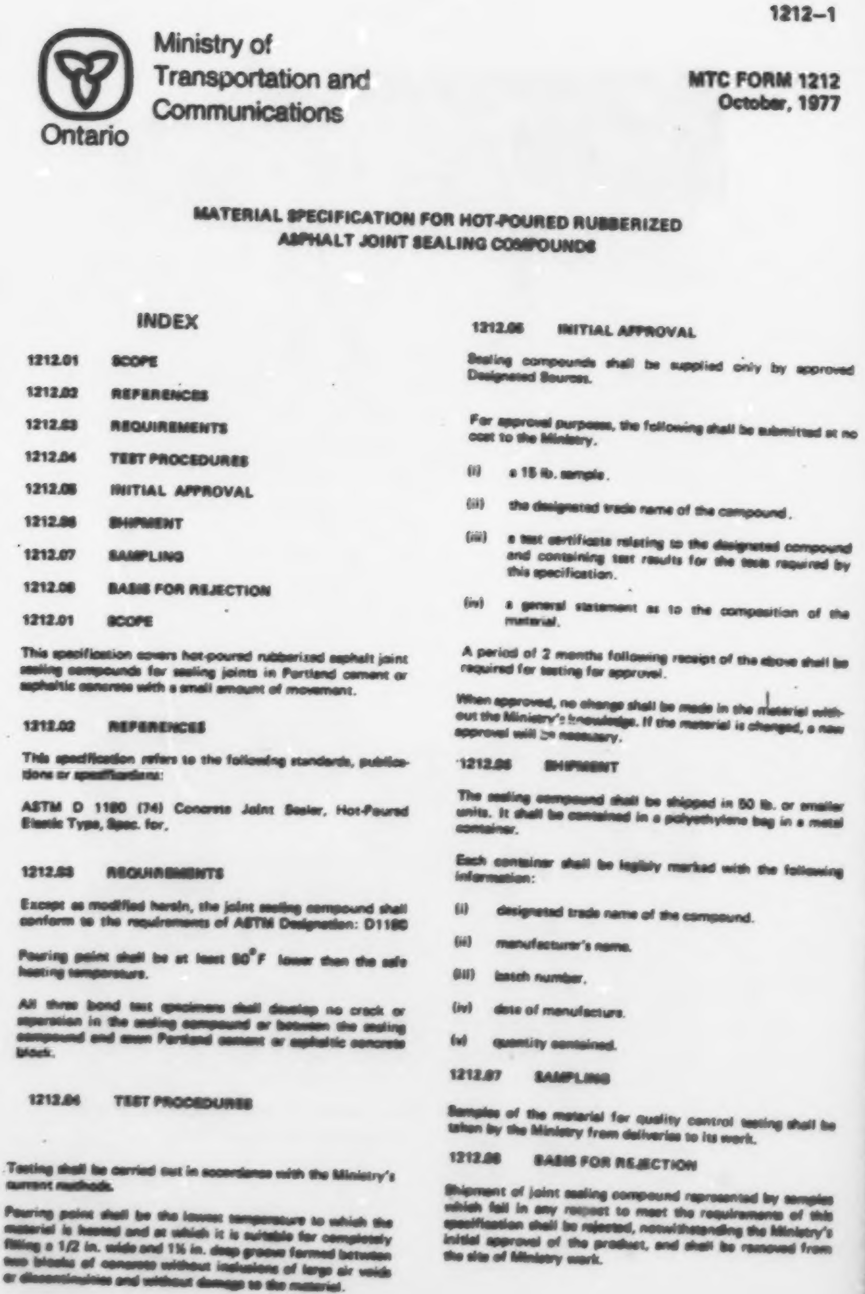


Figure 31/ Cone Applicator



1212–1

Ministry of
Transportation and
Communications
Ontario

MTC FORM 1212
October, 1977

MATERIAL SPECIFICATION FOR HOT-POURED RUBBERIZED ASPHALT JOINT SEALING COMPOUNDS

INDEX

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1212.01 SCOPE

This specification covers hot-poured rubberized asphalt joint sealing compounds for sealing joints in Portland cement or asphaltic concrete with a small amount of movement.

1212.02 REFERENCES

This specification refers to the following standards, publications or specifications:

ASTM D 1190 (74) Concrete Joint Sealer, Hot-Poured Elastic Type, Spec. for.

1212.03 REQUIREMENTS

Except as modified herein, the joint sealing compound shall conform to the requirements of ASTM Designation: D1190

Pouring point shall be at least 50°F lower than the safe heating temperature.

All three bond test specimens shall develop no crack or separation in the sealing compound or between the sealing compound and asphaltic concrete or Portland cement or asphaltic concrete block.

1212.04 TEST PROCEDURES

Testing shall be carried out in accordance with the Ministry's current methods.

Pouring point shall be the lowest temperature to which the material is heated and at which it is suitable for completely filling a 1/2 in. wide and 1½ in. deep groove formed between two blocks of concrete without inclusions of large air voids or discontinuities and without damage to the material.

1212.05 INITIAL APPROVAL

Sealing compounds shall be supplied only by approved Designated Sources.

For approval purposes, the following shall be submitted at no cost to the Ministry.

(i) a 15 lb. sample.

(ii) the designated trade name of the compound.

(iii) a test certificate relating to the designated compound and containing test results for the tests required by this specification.

(iv) a general statement as to the composition of the material.

A period of 2 months following receipt of the above shall be required for testing for approval.

When approved, no change shall be made in the material without the Ministry's knowledge. If the material is changed, a new approval will be necessary.

1212.06 SHIPMENT

The sealing compound shall be shipped in 50 lb. or smaller units. It shall be contained in a polyethylene bag in a metal container.

Each container shall be legibly marked with the following information:

(i) designated trade name of the compound.

(ii) manufacturer's name.

(iii) batch number.

(iv) date of manufacture.

(v) quantity contained.

1212.07 SAMPLING

Samples of the material for quality control testing shall be taken by the Ministry from deliveries to its work.

1212.08 BASIS FOR REJECTION

Shipment of joint sealing compound represented by samples which fail in any respect to meet the requirements of this specification shall be rejected, notwithstanding the Ministry's initial approval of the product, and shall be removed from the site of Ministry work.

Figure 32/ MTC Form 1212 Specification for Hot-Poured Sealant

Session 5

Low Temperature Properties of Some Selected Bituminous Mixes and Application to Design

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Hokkaido Institute of Technology

Ralph Haas
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University of Waterloo

LOW TEMPERATURE PROPERTIES OF SOME SELECTED BITUMINOUS MIXES AND APPLICATION TO DESIGN

by

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Hokkaido Institute of Technology

and

Ralph Haas,
Professor of Civil Engineering,
University of Waterloo

ABSTRACT

The problem of low-temperature cracking of asphalt pavements is severe in many parts of North America and several areas in Japan. As a consequence, researchers in Japan, Canada and the U.S. have carried out a major amount of work on this problem and have made considerable progress.

This paper reports on a combination of such progress made by several Japanese and Canadian researchers, including the authors. First, the key factors which can affect low-temperature cracking are identified and briefly described. They include the type and properties of the bitumen and the temperature conditions appropriate to the site.

A method is presented for characterizing the low-temperature properties of bituminous mixes using a "transition point", which is the peak of the flexural strength vs. temperature curve, represents an equivalent stiffness for all mixes, and represents the transition between brittle and ductile failure.

This transition point correlates well with certain common bitumen properties and this has been used to construct a nomogram whereby fracture temperature can be estimated from these properties.

A comparison has been made between such estimated cracking temperatures and observed cracking temperatures from several investigations. It was concluded that the estimation method provides reasonable answers for design purposes.

An equation for predicting frequency of low-temperature cracking as a function of bitumen stiffness, age of pavement, winter design temperature, subgrade soil type and thickness of pavement has also been presented.

Finally, some of the limitations of current technology have briefly been considered. They include an inability to adequately model all the variables affecting low-temperature cracking, and the need for conducting research on the cracking potential of recycled pavements.

1. INTRODUCTION

The performance of flexible pavements depends on a variety of construction, materials, traffic, maintenance and climatic factors. In cold areas, the climatic factors are especially important and can severely reduce the service life of a pavement.

Low-temperature transverse cracking (also known as thermal cracking, shrinkage cracking, and non-load-associated cracking) is one of the types of flexible pavements distress that can occur. It leads to significant losses in riding quality, to increased maintenance costs and to a resurfacing before the end of the normal design period. This type of cracking has become a serious and extensive problem in cold areas where winter temperatures drop below about -23°C.[1] It is extremely difficult, however, to define one single temperature at which cracking will occur, because it depends on a number of factors. For example, thermal cycling, in combination with other factors, may play a significant role in the fact that such cracking even occurs extensively as far south in the United States as West Texas and New Mexico.

Low-temperature transverse cracking was recognized in the early to mid 1930's, but the problem was not too acute. Because traffic volumes and construction expanded rapidly after World War II, and the performance requirements for pavements increased, a number of highway engineers became concerned in the late 1950's and early 1960's. It was explicitly recognized as well in an A.A.P.T. symposium session in 1966.[2] The Ste. Anne Test Road was built in 1967, and many of the design criteria being proposed have been developed from data obtained from this test road.[3,4,5,6] Two comprehensive reports on Design Method to minimize low-temperature transverse cracking have been published.[1,7]

This paper has as its general purpose the description of low temperature properties of some selected bituminous mixtures and how this information can be used in design. More specifically, the major objectives are as follows:

1. To describe some of the major variables influencing pavement transverse cracking.
2. To describe a practical method for characterizing the low-temperature properties of the bituminous component.
3. To provide some actual properties of a range of bituminous mixes and to illustrate how these may be used in a design sense.
4. To illustrate how low-temperature transverse cracking frequency can be predicted.
5. To discuss the limitations of current low-temperature technology.

2. VARIABLES INFLUENCING PAVEMENT TRANSVERSE CRACKING

A number of factors that can effect low-temperature transverse cracking have been presented and discussed.[1,7] The situation in nature is complex with a large number of factors and their interaction involved, such as restraint conditions, temperature, subgrade and pavement components. It must be recognized that fracture in a bituminous surface can occur due to causes other than low-temperature shrinkage. These causes can be related to the effects of asphalt aging or other short and long term mixture properties.

Figure 1[8] attempts to summarize the variety of bituminous mixture properties and all other possible factors that may be significant to low-temperature transverse cracking. This listing is subdivided into external and component factors.

A number of factors are known to influence the extent and severity of bituminous pavement transverse cracking such as, climatic effects, pavement design and structural effects, asphalt properties, mixture properties and mixture design, age of pavement, subgrade type, etc. The rheological properties of asphalts have been considered as a major variable influencing pavement transverse cracking and many studies on relationships between asphalt properties and field measurement of cracking have been pub-

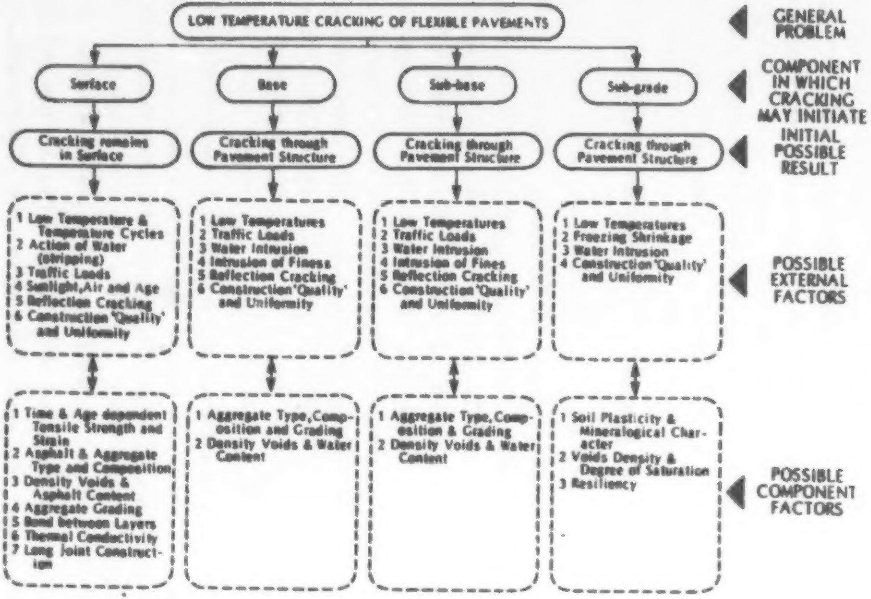

LOW TEMPERATURE CRACKING OF FLEXIBLE PAVEMENTS
GENERAL PROBLEM
Surface
Base
Sub-base
Sub-grade
COMPONENT IN WHICH CRACKING MAY INITIATE
Cracking remains in Surface
Cracking through Pavement Structure
Cracking through Pavement Structure
Cracking through Pavement Structure
INITIAL POSSIBLE RESULT
1 Low Temperature & Temperature Cycles
2 Action of Water (stripping)
3 Traffic Loads
4 Sunlight, Air and Age
5 Reflection Cracking
6 Construction 'Quality' and Uniformity
1 Low Temperatures
2 Traffic Loads
3 Water Intrusion
4 Intrusion of Fines
5 Reflection Cracking
6 Construction 'Quality' and Uniformity
1 Low Temperatures
2 Traffic Loads
3 Water Intrusion
4 Intrusion of Fines
5 Reflection Cracking
6 Construction 'Quality' and Uniformity
1 Low Temperatures
2 Freezing Shrinkage
3 Water Intrusion
4 Construction 'Quality' and Uniformity
POSSIBLE EXTERNAL FACTORS
1 Time & Age dependent Tensile Strength and Strain
2 Asphalt & Aggregate Type and Composition
3 Density Voids & Asphalt Content
4 Aggregate Grading
5 Bond between Layers
6 Thermal Conductivity
7 Long Joint Construction
1 Aggregate Type, Composition and Grading
2 Density Voids & Water Content
1 Aggregate Type, Composition & Grading
2 Density Voids & Water Content
1 Soil Plasticity & Mineralogical Character
2 Voids Density & Degree of Saturation
3 Resiliency
POSSIBLE COMPONENT FACTORS

Figure 1. Factors of possible significance in low-temperature cracking of flexible pavements. After [8].

lished.[9,10,11,12] Although it is difficult to quantify the other factors, it is necessary to clarify effects of these factors on the pavement transverse cracking.

Climatic Effects

The pavement surface temperature which directly influences pavement transverse cracking is obviously dependent on the ambient air temperature. The relationship between the minimum ambient air temperature and the minimum pavement surface temperature at the depth (6 mm to 13 mm) is shown in Fig. 2.[13]

Pavement Thickness

Thin asphalt concrete pavements are less resistant to transverse cracking than full-depth asphalt pavements.[3,7] The transverse cracking is usually more severe in the thinner pavement sections relative to thick pavements of otherwise similar construction.

Subgrade

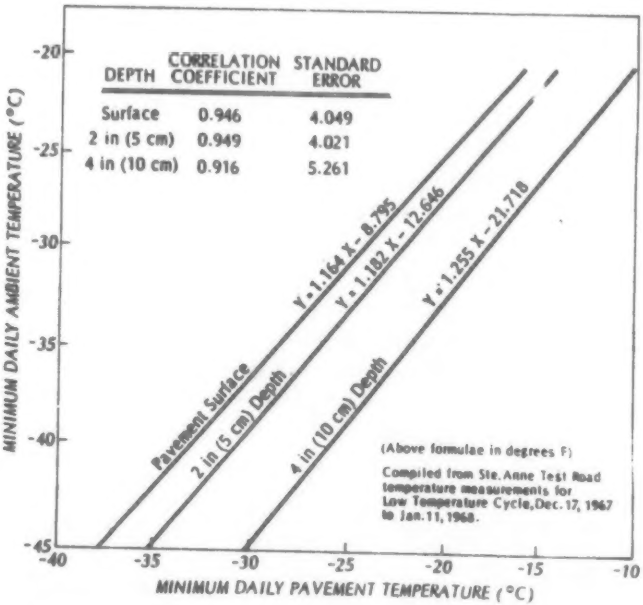
The results of the St. Anne Test Road which were built over both sand and clay subgrades showed that subgrade type had a very considerable influence on the frequency of low-temperature transverse cracking. This frequency is usually considerably higher for pavements constructed on sand subgrades, all other variables being constant.[14,15] The effect of subgrade soil type on a theoretical basis of tensile forces at low temperature may be related to water held in the soil.[16]

Age of Pavement

It is recognized that the age of pavement affects the frequency of low-temperature transverse cracking, because the cracking frequency increases with time in service for pavements already cracked. It is considered that the low-temperature transverse cracking will occur under the repeated changes of pavement temperature.

Asphalt Properties

The relationship between asphalt properties and the occurrence of pavement transverse cracking has been apparent for some years. The properties of asphalt have been evaluated by means of stiffness, viscosity, penetration, ductility, etc. Stiffness of asphalt at low temperature has been



DEPTH	CORRELATION COEFFICIENT	STANDARD ERROR
Surface	0.946	4.049
2 in (5 cm)	0.949	4.021
4 in (10 cm)	0.916	5.261

(Above formulae in degrees F)

Compiled from Ste. Anne Test Road temperature measurements for Low Temperature Cycle, Dec. 17, 1967 to Jan. 11, 1968.

Figure 2. Relationship between minimum daily ambient temperature and corresponding pavement temperature during a low temperature. After [13].

demonstrated[3,7,9,17] to correlate with pavement transverse cracking. Consequently, asphalt stiffness, or other properties that can be related to stiffness, have served as the basis for making a choice of asphalt in mixture design at low temperatures.

It is generally agreed that stiffness should be defined as the (secant) modulus of the stress-strain relationship for a bituminous material under the particular conditions of temperature and time of loading involved; i.e.

$$S(t,T) = \frac{\sigma}{\varepsilon}$$

where $S(t,T)$ = stiffness modulus for temperature T and time of loading, t

σ = stress at t and T

ε = unit strain at t and T

Mix Properties

A number of factors such as asphalt content, degree of compaction, air voids, and aggregate gradation are known to influence the mechanical properties of asphalt mixtures (such as stiffness of mixture, fracture strength, etc.). Consequently, these factors might be expected to have some influence on low-temperature transverse cracking. According to the results of the St. Anne Test Road,[3] however, it was demonstrated that the asphalt content and the resulting air voids had no influence on low-temperature transverse cracking.

The degrees of compaction of mixture has an influence on strength and stiffness of mixture[18]. Absorption of asphalt into aggregate will have an influence on mix stiffness[19]. The influence of these factors on pavement transverse craacking have not been studied in detail.

3. LOW-TEMPERATURE PROPERTIES OF BITUMINOUS MIXTURES

Stress-Strain Relationship

In general, the relationships between stress and strain of a bituminous mixture at lower temperature and varying rate of strain can be divided into the following classes[20], as shown in Figure 3:

Type I : Elastic (or Brittle) failure

Type III : Viscoelastic (or Ductile) failure

Type II : Failure at Transition zone between type 1 and III

The bituminous mixture generally shows one of the three types of response under specific conditions and in accordance with the change of external conditions. Under different conditions, different responses are shown. It should be recognized that the above external condition may occur in very common situations.

Flexural Strength, Strain, Stiffness

It appears that the fracture mode, breaking strength and strain at failure of bituminous mixtures also depend on both temperature and loading time. These have been obtained by various test methods, including a flexure test[20]. The dimension of specimen was 25 mm x 25 mm x 250 mm, and the three point bending in a span length of 200 mm was employed. Load and deflection were measured by a load cell and L.V.D.T. Stresses and strains are calculated with an elastic solution.

The behavior of asphalt concrete as regards flexural strength, strain at failure and stiffness at failure at various temperatures with respect to applied rate-of-strain is shown in Fig. 4.

The peak point of the flexural strength-temperature curve was termed "transition point" by T. Sugawara.[21] On the lower temperature side from the point, the stress-strain curve tends to show Type I behavior and the fracture mode is brittle. On the higher temperature side from the point, it shows visco-elastic properties and the stress-strain curve may belong to Type III.

Transition Point

The peak of the flexural strength-temperature curve would not be a point in practical terms but the zone of a certain narrow range around the peak. The stress-strain curve is somewhat similar to that of elastic materials. This narrow zone can be represented by a stress-strain curve of Type II. The transition point (Tt) may move to the lower or higher temperature side depending on the variables shown in Figure 5.

a) Effect of Rate-of-Strain

Fig. 6 shows the effect of rate-of-strain on the movement of the transition point. A semi-logarithmic relationship was obtained in various types of bituminous mixtures. The difference for the range of rate-of-

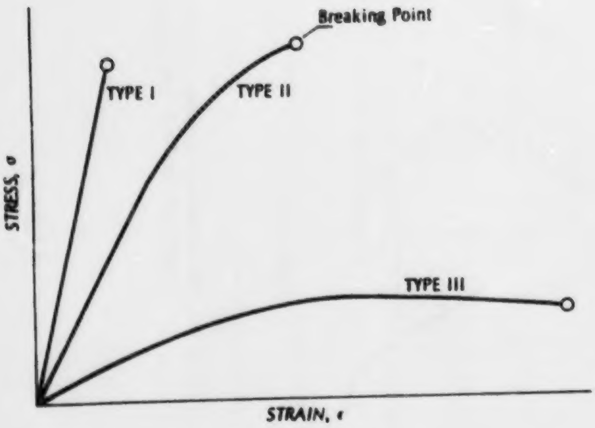


Figure 3. Typical relationship between stress and strain of a bituminous mixture.

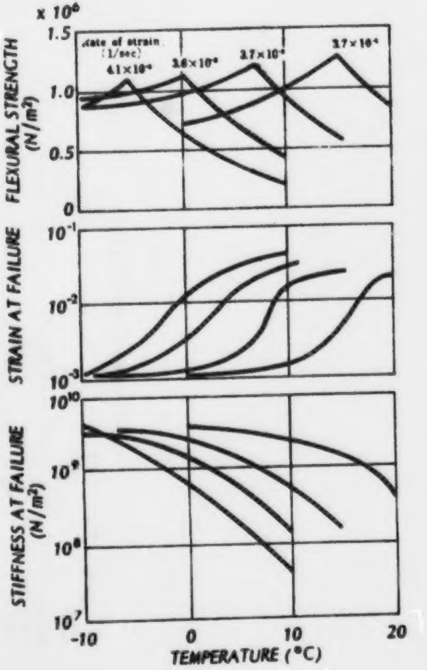


Figure 4. Effect of rate-of-strain on flexural strength, strain at failure and stiffness at failure of asphalt concrete. After [20]

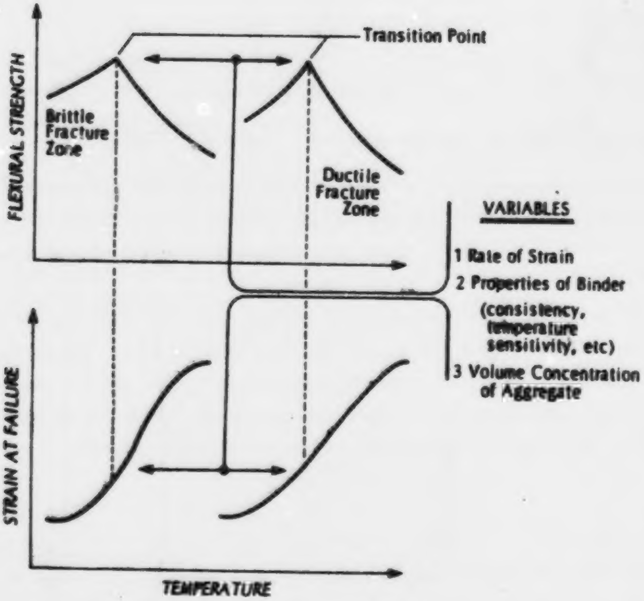

Transition Point
FLEXURAL STRENGTH
Brittle Fracture Zone
Ductile Fracture Zone
VARIABLES
1 Rate of Strain
2 Properties of Binder (consistency, temperature sensitivity, etc)
3 Volume Concentration of Aggregate
STRAIN AT FAILURE
TEMPERATURE

Figure 5. Relationship between flexural strength, strain at failure and temperature.

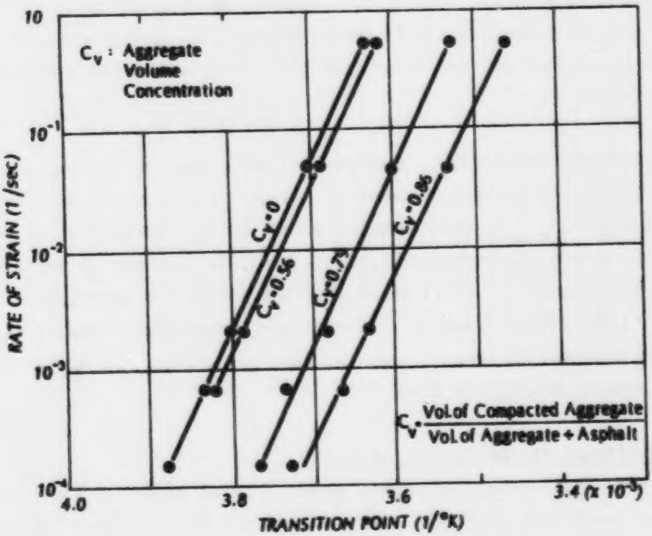


Figure 6. Effect of rate-of-strain on movement of transition point. After [33].

strain for any one curve in Figure 6 is about 6 ± 1°C from the transition point.[33]

b) Effect of Bituminous Properties

Various bitumens of different rheological properties were tested in asphalt concrete by T. Sugawara and A. Kasahara[22] in order to obtain the transition points. The bituminous properties are given in Table 1 and the composition of the asphalt concrete is shown in Table 2.

A strong correlation was observed between Penetration Index (P.I.) of bitumens and the temperature differences between the softening point ($T_{R.\&B}$) of bitumen and the transition point (Tt) of asphalt concrete as shown in Figure 7.

Based on the foregoing, a nomogram was developed for determining a transition point of asphalt concrete as illustrated in Figure 8. If the softening point ($T_{R.\&B}$) and the penetration (25°C, 100 g, 5 sec.) of bitumen are known, the transition point of asphalt concrete is simply obtained from Figure 8. It is shown that the larger the penetration of asphalt (25°C, 100 g, 5 sec.) at the same penetration index or the higher the penetration index at the same penetration, the transition point of asphalt mixture is lower.

Mixture Stiffness at the Transition Point

The flexural strength of normal paving asphalt concrete (Cv = 0.86) around the transition point can reach about 1.2 x 10^7 N/m^2 and the strain-at-failure at the transition point is about 500 x 10^{-6}. Consequently, stiffness of the asphalt concrete at the transition point is about 2.4 x 10^9 N/m^2. It is recognized that the value will be independent of the applied rate-of-strain and asphalt properties for the asphalt concrete used in asphalt pavement. Therefore it is considered that the transition point represents an equivalent stiffness temperature for brittle fracture.

4. PREDICTING PAVEMENT CRACKING TEMPERATURE

Many procedures have been developed for predicting pavement cracking temperature from both asphalt and mixture stiffness. A listing of the procedures is as follows[1]:

Table 1

BITUMINOUS PROPERTIES

		Penetration (100g, 5sec, 25°C)	Softening Point $T_{R\&B}$ (°C)	Penetration Index	Viscosity (poises)			Equi-Viscous Temperature (°C)		
					at 45°C	at $T_{R\&B}$	at 60°C	Tequi.[1]	Mixing Temp.[2]	Rolling Temp.[3]
STRAIGHT	ST-1	50	51.0	-1.0	5.56×10^4	1.81×10^4	4.85×10^3	50.8	160	112
ASPHALT	ST-2	72	48.0	-0.9	2.89×10^4	1.75×10^4	2.94×10^3	47.0	154	105
	ST-3	98	44.0	-1.2	1.42×10^4	1.56×10^4	1.50×10^3	43.0*	151	102
SEMI-BLOWN	SB-A-1	36	57.0	-0.4	1.51×10^5	2.09×10^4	1.36×10^4	57.2	175	119
ASPHALT	SB-A-2	50	56.5	+0.3	1.14×10^5	1.40×10^4	7.95×10^3	54.4	156	110
TYPE A	SB-A-3	50	54.0	-0.3	6.69×10^4	1.65×10^4	5.23×10^3	52.5	158	109
	SB-A-4	52	52.0	-0.7	5.50×10^4	1.70×10^4	5.15×10^3	50.8	162	113
	SB-A-5	67	50.0	-0.5	3.26×10^4	1.13×10^4	2.15×10^3	47.5	144	101
	SB-A-6	69	50.5	-0.3	3.24×10^4	1.16×10^4	2.58×10^3	47.5	148	98
	SB-A-7	71	51.0	-0.1	2.89×10^4	1.03×10^4	2.46×10^3	47.0	156	105
	SB-A-8	71	57.0	+1.3	1.43×10^3	1.39×10^4	8.13×10^3	54.8	156	110
	SB-A-9	72	50.5	-0.1	3.42×10^4	1.36×10^4	2.68×10^3	47.7	153	106
	SB-A-10	72	52.0	+0.2	5.02×10^4	1.52×10^4	4.02×10^3	50.2	159	110

[1] 2×10^4 poises [2] 2 poises [3] 30 poises * by extrapolated

Table 2

COMPOSITION OF ASPHALT CONCRETE

Coarse aggregate (max. size: 13 mm)	49.9%
Fine aggregate (max. size: 2.5 mm)	37.3%
Mineral filler (limestone)	6.8%
Asphalt content	6.0%
Air voids	3.0%

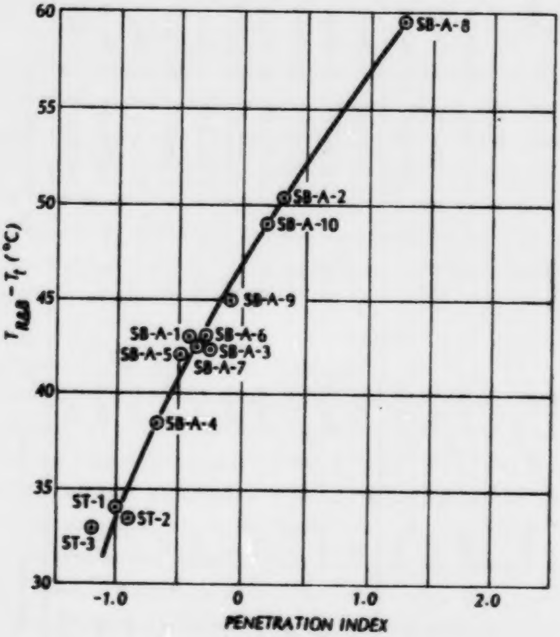


Figure 7. Relationship between P.I. of bitumens and temperature difference between softening point of bitumen and transition point of asphalt concrete.

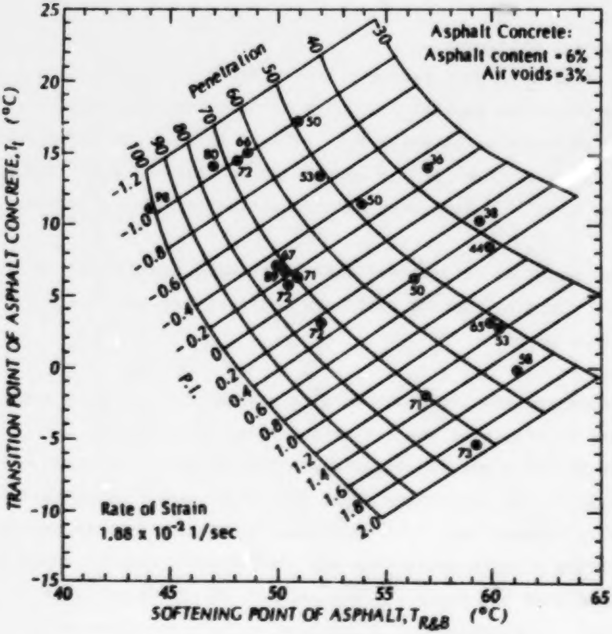


Figure 8. Nomogram for determining the transition point of asphalt concrete.

A. Asphalt Stiffness Procedures
    (1) Limiting Asphalt Stiffness[3,23,24]
    (2) Critical Asphalt Stress[25]
    (3) Nomographic Cracking Temperature[6,25]
B. Mixture Stiffness Procedures
    (1) Limiting Mix Stiffness[26,6]
    (2) Calculated Mix Cracking Temperature[11]
C. Direct Mix-Cracking Observations[8,27,25,28,29]

Limiting Asphalt Stiffness

It is considered that pavement transverse cracking will occur at the temperature which the asphalt reaches a certain limiting stiffness. Several values of asphalt stiffness such as $2. \times 10^8$ N/m^2 at 2 hours loading time[30,31], 1.4×10^8 N/m^2 at 2.8 hours loading time,[7] 1×10^9 N/m^2 at half-hour loading time[23,24] have been adopted as the limiting stiffness.

In this paper it is proposed that a cracking temperature can be predicted with the limiting asphalt stiffness obtained from mixture stiffness at the transition point.

The necessary steps for determining a predicted cracking temperature from mixture stiffness at the transition point then become:

1. Determine a corresponding asphalt stiffness to mixture stiffness at the transition point (2.4×10^9 N/m^2) using relationship between moduli of stiffness of asphalt cements and of paving mixture containing the same asphalt cements based on Heukelom and Klomp.[32] The asphalt stiffness of 1.4×10^7 N/m^2 is determined as a limiting asphalt stiffness which will occur at brittle fracture (for Cv = 0.86).
2. Read each temperature difference on Van der Poel's nomogram, corresponding to the asphalt stiffness of 1.4×10^7 N/m^2 and 10-hour loading time which is considered as a cooling time duration in the field.

The relationship between softening point ($T_{R.\&B}$) of asphalt, and predicted pavement cracking temperature is illustrated in Figure 9. If the asphalt properties such as softening point ($T_{R.\&B}$), Penetration (25°C, 100 g, 5 sec.) and Penetration Index are known, the predicted pavement cracking temperature of asphalt concrete is simply obtained from Figure 9.

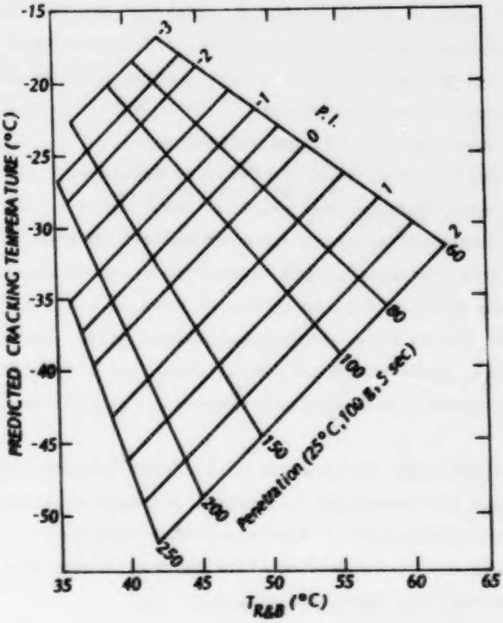


Figure 9. Nomogram for predicting cracking temperature from softening point, penetration and P.I. of the asphalt.

Correlation Between Predicted and Observed Cracking Temperature

There is little data of observed cracking temperature[6,13,17,24] in the field because of the difficulty of accurate measurements of both time of cracking and temperature.

Fig. 10 shows the relationship between the predicted cracking temperature by the method of this paper and observed cracking temperature in the field, or minimum pavement temperature in the case of no-cracking, together with direct observations of temperature of mixture cracking in the laboratory.[8,29] It seems that the predicted cracking temperatures are reasonable, but the temperature may be higher by about 4°C than several observed cracking temperatures in the field and direct observations of temperature of mixture cracking in the laboratory.

These reasons can include the following:

1. A number of additional factors, as explained previously, can influence the observed cracking temperature in the field.
2. The predicted cracking temperatures are obtained with the mixture stiffness at the transition point as determined from flexural tests instead of direct tensile testing.
3. There can be many differences between mixes, such as asphalt source, asphalt content, etc., which can influence the difference between predicted and observed cracking temperatures.

Nevertheless, Fig. 10 does illustrate that cracking temperature predictions, using the method of this paper, are generally within a reasonable deviation from observations. Thus the method represents a useful design for quickly and easily estimating cracking temperature based on simple and routinely determined bitumen properties.

5. PREDICTING THE AMOUNT OF CRACKING

When a prediction has been made that a mix will crack (according to Sec. 4), the next desirable step would be to determine the amount of cracking that might be expected to occur.

A mathematical model for estimating such frequency of cracking (using regression techniques) was originally developed by Hajek (16) and further described by Haas (7). The form of the model chosen after evaluating a large number of alternative forms, is as follows:

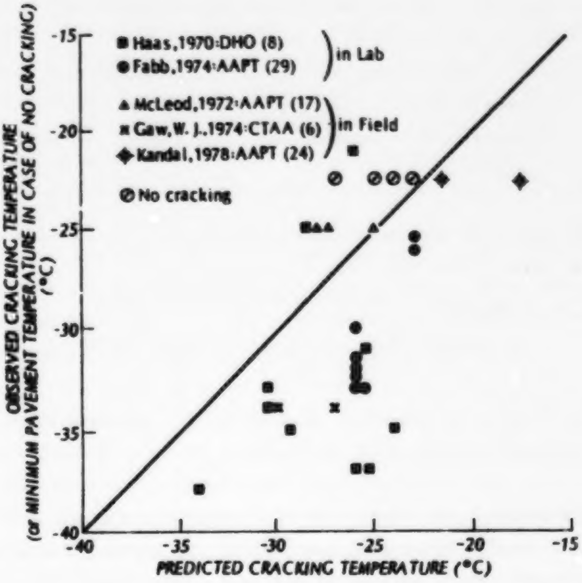


Figure 10. Observed cracking temperature vs. predicted cracking temperature, from several investigations.

$$10^{I} = c_1 S^{(C_2+C_3+C_4 a)} C_5{}^{d} D_6{}^{m} d^{C1S}$$

where

I = cracking index (number of full plus one half of the half transverse cracks per 500 ft. section of 2-lane road) as originally defined by Ontario's former Dept. of Highways.

S = stiffness of the original asphalt cement in kg/cm^2, determined by McLeod's method for a loading time of 20,000 seconds and for the winter design temperature.

t = combined thickness of the bituminous layers in inches,

a = age of the pavement, in years,

m = winter design temperatures, in °C, obtained from a relationship between m and air freezing index (see Ref. 7)

d = subgrade type (dimensionless code: 5-sand, 3-loam, 2-clay)

C_1-C_7 = constants.

The constants were established from regression analysis and the final, numerical form of the model is:

$$10^{I} = 2.497\cdot 10^{30} \times S^{(6.7966 - 0.8740t + 1.3388a)}$$
$$.(7.054\cdot 10^{-3})^{d}\cdot(3.193\cdot 10^{-13})^{-0.1m}\cdot d^{0.602655}$$

The R^2 value of the model is 0.82 and the standard error of estimate of the dependent variable, I, is 6.2.

If a designer concluded from application of the model that the cracking frequency was unacceptably high, the following alternatives could be considered: (a) a softer, higher penetration grade asphalt, (b) a less-temperature susceptible asphalt, or (c) a soft asphalt with an additive such as sulphur to increase high temperature stiffness. Of course other requirements, such as fatigue life, rut depth development, etc. would have to be checked.

There are several limitations to the model and these include the following. (a) It may not represent the "true law" which governs low-temperature cracking frequency, and it undoubtedly does not incorporate all the variables which may be important for a particular field situation. (b) Extrapolation outside the range of values used for the independent variables in developing the model is risky. (c) It was assumed that the cracks observed in the field to measure cracking index were caused by low temperatures, but it is quite possible that some were the result of other causes.

6. SOME CURRENT PROBLEMS AND RESEARCH NEEDS

There are some major problems still in existence, and some major research needs. They include the following.

1. Current technology is not able to directly measure cracking potential of the binder itself, although some attempts have been made at developing equipment. Instead, surrogate values of viscosity-temperature, penetration, $T_{R.\&B}$, etc. are used. It would be desirable to be able to directly test any given asphalt, under simulated conditions of the project in which it is to be used, for low-temperature cracking resistance. A reliable test could also be used for product specification purposes.
2. The actual low-temperature cracking phenomenon in the field, and the frequency of cracking, is very complex. While the bituminous component is a major variable, other variables can be important. Current technology is not yet able, however, to adequately model the entire range and interaction of variables.
3. Current methods are only able to consider new pavements. When pavements have already cracked, and this is the case for much of the paved network in existence today, we face a major problem of reflection cracking through overlays. The only alternative is to completely remove the bituminous layer. Rubber-asphalt interlayers, between the old, cracked surface and the overlay, may provide an answer but their performance and cost-effectiveness for cold climates is still to be demonstrated.
4. For the situation where an old cracked layer is completely removed and the mix is recycled, with a recycling agent or "rejuvenator", which is becoming increasingly popular, the question exists as to whether these new mixes will crack. Very little research has been done on rejuvenated binders and/or recycled mixes for low-temperature cracking resistance.

7. CONCLUSIONS

The major points of this paper may be summarized as follows.

1. Low-temperature transverse cracking of asphalt pavements is influenced by a large number of variables, including type and properties of the bitumen used, climate conditions, subgrade types, and pavement components. Of these, the combination of bitumen and temperature has the major effect, but the actual phenomenon of cracking is complex and other variables can also be important in any given situation.
2. The low-temperature properties of bituminous mixtures can be characterized by their stress-strain-time-temperature response. One such characterization is a "transition point", at the peak of the flexural strength vs. temperature curve. A strong correlation exists between the penetration index, P.I., of the bitumen and the temperature difference between the ring and ball softening point of the bitumen, $T_{R\&B}$, and the transition point temperature. This has enabled construction of a nomogram whereby the transition point of the mix can be simply determined from P.I., $T_{R\&B}$ and penetration of the asphalt.
3. Considering the transition point as representing an equivalent stiffness for brittle fracture, another nomogram has been developed to fracture temperature from $T_{R\&B}$, P.I. and penetration.
4. This nomogram in 3. has been used to compare estimated fracture temperatures with observed cracking temperatures from several sources. The results show that the nomograph can provide reasonable estimates for design purposes.
5. An equation for predicting low-temperature cracking frequency as a function of bitumen stiffness, age of pavement, type of subgrade, pavement thickness and winter design temperature has also been presented.
6. Low-temperature technology still has several limitations. These include an inability to adequately model all the variables which affect low-temperature cracking. As well, little research has yet been conducted on low-temperature properties of recycled mixes.

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Session 5

Study on the Abrasion Problem in Japan

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STUDY ON THE ABRASION PROBLEM IN JAPAN

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ABSTRACT

The inhabitants of Japan are allowed to equip their vehicles with studded tires and, consequently, a serious pavement abrasion problem occurs in winter. A study of this problem was conducted.

The first part of this paper describes the estimation method for pavement abrasion which was made by observing results on real roads from 1977 to 1979. The next part considers the anti-abrasion effect of asphalt binder containing additives, such as rubber, epoxy resin, etc. by using test pavements on national highways. The last part compares the abrasion test methods used in Japan, especially at the regional offices of the Ministry of Construction.

PREFACE

The inhabitants of Japan are allowed to equip their vehicles with studded tires and, consequently, a serious pavement abrasion problem occurs in winter. Research into this problem was carried out.

This paper describes the research on the actual condition of abrasion and the prediction method of wear using data such as traffic, weather conditions, etc. The next part reviews the anti-abrasion effects of asphalt binders to which additives such as rubber, epoxy resin, etc. have been added. The results are from test pavements on actual highways. The last part is a report on results of a joint test conducted in order to compare the various types of ravelling tests used for estimating the wear resistance of asphalt mixtures.

Thus, 1) the comprehensive consideration of the actual depth of wear and the prediction method for national highway pavements in Japan, 2) the development of anti-wearing mixtures for pavement and 3) the estimation of testing methods are introduced.

1/ ACTUAL CONDITIONS OF ABRASION AND THEIR ESTIMATION

1.1/ Research Method

Research on the actual conditions of abrasion and depth of wear in representative parts of a snowy and cold area was carried out as follows:

- formation of the research schedule of actual condition of abrasion, summarizing of the research outline and sending measuring pins as control points for measuring depth of wear to respective prefectures;
- selecting the measuring points according to the outline of research and burying the measuring pins;
- carrying out the research on depth of wear and the research concerned during three years; and
- summarizing the data and analyzing it.

The research schedule of the actual condition of abrasion consists of Form 1 to Form 5 and the outline of the Forms are as follows.

Form 1: Table showing measuring points, name of route, thickness and material of respective layers of pavement and buried position of measuring pins.

Form 2: Table with types and thicknesses of asphalt concrete of wearing layer, traffic volume in autumn and winter, frequency of use of each type of tire in winter traffic (measured data of frequency in use of each type of tire in traffic volume corresponding to about 10% of winter traffic volume), actual measured data of depth of wear, snow removal days, and days of using snow extinguishing pipe. (Snow removal days and snowfall days are obtained from the days recorded by the road information monitor or patrol during a month.)

Form 3: Figure VI showing cross-sectional unevenness (data which measured cross-sectional unevenness each 10 cm interval erecting staffs on measuring pins).

Form 4: Data table of cross-sectional unevenness.

Form 5: Photograph album.

The measuring pin is made of aluminum having the shape shown in Figure 1 and and was introduced by the Hokkaido Development Bureau, Civil Engineering Research Institute. The diameter of the measuring pin tip is fixed at 10 cm in order to obtain the depth of wear by measuring the diameter. These pins, three horizontally at 1.5 m intervals and three longitudinally at 3 m intervals, were buried in the road at designated control points.

The depth of wear is the average unevenness of three measuring lines in longitudinal section and is obtained by averaging the down slopes of Figure VI showing a cross-sectional unevenness of Form 3. The maximum is the one which has the largest down grade.

As a result of the research carried out from 1977 to 1980, the actual conditions of abrasion of 28 roads in 13 prefectures were studied as is shown in Figure 2.

1.2/ Results of Research

An example of the research results is given in Table 1 and the summarized results of the three years, 1977, 1978 and 1980 are given in Table 2.

The number of research points (n) was 74 (24 points x 3 years).

The term of research is the number of days regarded as being winter which is, on average, 120 days, but occasionally there are regional differences.

The lowest temperature recorded was -20°C in the data of 1977, but the mean value during the three years, ($\bar{x}$), was -9.0°C. The mean temperature ($\bar{x}$) was +1.3°C.

Snow removal days are, on average, 72 days or 60% of the winter term. It is supposed that the high percentage was caused by little snow during three years research.

The maximum freezing index, 461 (degree-days), was observed at two points in Hokkaido while, on the other hand, zero degree-days was recorded in Niigata, Toyama, Fukui, Shimane, etc.

The researched road was mainly for two traffic lanes, traffic division B (40%) and road traffic division C (33%), totalling about 80%, since this research was carried out mainly on regional (prefectural) highways.

In autumn, the total (entire) traffic volume ($\bar{x}$) was 4,900 (vehicles/day/direction) and the traffic volume of heavy vehicles ($\bar{x}$) was 648 (vehicles/day/direction); in winter, the total traffic volume ($\bar{x}$) was 4,100 (vehicles/day/direction) and the traffic volume of heavy vehicles ($\bar{x}$) was 520 (vehicles/day/direction). Consequently, it was observed that the traffic volume in winter is less by about 20% than that of autumn.

The number of wheels is calculated from the following formulae.

The total number of passed wheels = total traffic volume x 2
+ triaxial wheel traffic volume (wheels/day/direction).

The total number of heavy wheels = heavy vehicle traffic volume x 2
+ triaxial wheel traffic volume (wheels/day/direction).

The equipped rate of tire chains (rate of vehicles equipped with tire chains) was calculated by multiplying the number of the passed wheels by the rate of the number of wheels for each type of tire, researching the equipped condition of traffic volume corresponding to about 10% of the traffic volume in winter (vehicles/day/direction). However, the equipped rate of tire chains was modified as follows. It was obtained by the following formula by regarding vehicles not equipped with snow tires or studded tires as equipped with chains:

TC = 100 - (SP + SN) (%)

where, TC = Equipped rate of tire chains
SP = Equipped rate of snow tires
SN = Equipped rate of studded tires

With the formula above, the equipped rate of snow tires was obtained as $\bar{x}$ = 39% for the total traffic volume and $\bar{x}$ = 53% for heavy vehicle traffic volume. Regions having a high rate of vehicles equipped with snow tires (over 70%) for both total traffic volume and large vehicle traffic volume were Yamagata, Fukushima, Niigata, Toyama, Gifu and Fukui.

The equipped rate of studded tires was $\bar{x}$ = 47% for the total traffic volume and $\bar{x}$ = 28% for heavy vehicle traffic volume and so, it was observed that studded tires were widely spread and that there was a regional difference because the regions with a particularly high studded tire rate (over 70%) were Hokkaido, Aomori and Nagano.

The equipped rate of tire chains was $\bar{x}$ = 7.4% for the total traffic volume and $\bar{x}$ = 10.0% for heavy vehicle traffic volume, and it was observed that the frequency in the use of tire chains was small compared with that of studded tires.

The maximum depth wear ranged from 1.0 mm to 20.0 mm and a wide dispersion was observed. Regions of large wear depth (over 8 mm) were in northern Japan; on the other hand, regions of small wear depth (under 4 mm) were in central Japan. The mean wear depth ($\bar{x}$) was 2.5 mm; the 90% reliable section was 1.2 to 3.8 mm. Therefore, the mean winter road wear depth could be assumed to be about 3.0 mm.

Table 3 was obtained by researching the depth of wear for each type of surface mixture. Even though the data were few, it was observed that frequencies of use of fine-graded asphalt concrete and fine- and gap-graded asphalt concrete were high in snowy regions. Examining the mean depth of wear for each type of mixture including dispersions, it can be observed that fine-graded asphalt concrete (13F) has a higher wear resistance — about 1.5 times that of fine-graded asphalt concrete and fine- and gap-graded asphalt concrete.

1.3/ Consideration

1.3.1/ Factors Concerning Abrasion

Factors concerning abrasion were examined by researching the total traffic volume, large vehicle traffic volume, relationships between cumulative values of each type of tire and cumulative maximum depth of wear and the cumulative mean depth of wear. An example of the results is given in Table 4.

The cumulative traffic volume {100 - (SP + SN)} was included in the factors in order to make an examination concerning the traffic volume of TC, as mentioned above. (100 - SP) was used for examination by the raised index from {100 - (SP + SN)} assuming that vehicles other than those with studded tires used tire chains in the case of examination of cumulative traffic volume and the depth of wear on days of snow or road freezing.

Consequently, the maximum wear depth was determined as being the wear depth shown by the following analyses. Considering the maximum wear depth shows a high correlation coefficient, the wear depth can be assumed by a good interrelationship betweeen maximum wear depth and mean wear depth as shown in Figure 3, and so maximum wear depth appears in problem points of the site.

In singular correlation, it was observed that the coefficient of correlation between the factors of winter days, snowy days, heavy vehicle traffic volume and traffic volume of vehicles equipped with studded tires and wear depth is high. This means that abrasion goes on in long winter regions where frequency in the use of studded tires is high.

The reason for the high correlation coefficient is the multiplying effect of the heavy vehicles equipped with studded tires and also the high proportion of the total traffic that is equipped with studded tires. As considered above, it was observed that it was important to measure the traffic volume of vehicles equipped with studded tires and total traffic volume accurately in order to predict the depth of wear of roads.

On the other hand, the occupied frequency of traffic volume of vehicles equipped with tire chains for the total traffic volume was less than the traffic volume of vehicles equipped with studded tires, and the coefficient of correlation was low. Therefore, it is dangerous, for accuracy sake, to assume the depth of wear by only considering the traffic volume of vehicles equipped with tire chains.

In researching the relationship between various types of traffic volume and wear depth in winter, three examples of high correlation coefficients were found and are shown in Figures 4 to 6. The solid line shows a regression curve and the broken line shows an 80% reliable section in these Figures, and so depth of wear will be predicted by using the traffic volume of vehicles equipped with studded tires and the cumulative traffic volume in winter, as shown in these Figures. A result contrary to the prediction, with cumulative traffic volume of heavy vehicles having much more effect than the cumulative total traffic volume, was found (comparing Figures 5 and 6).

This was probably because this research was carried out mainly on roads of low percentage large vehicle traffic volume (about 15% on average). Therefore, it is assumed that the traffic volume of heavy vehicles may greatly affect road wear.

The relationship between Figure 7 and Figure 8 was obtained by arranging the traffic volume of vehicles equipped with studded tires (a high correlation coefficient during winter period) and traffic volume of vehicles equipped with studded tires during snowfall days and snow removal days. It was assumed that the traffic volume of vehicles equipped with studded tires during snowfall days and snow removal days may be used for measuring the wear depth.

Figures 9 and 10 show the relationship between modified traffic volume of vehicles equipped with tire chains and wear depth. In each Figure, the correlation coefficient (R) increased about R = 0.08 over the one before modifying the traffic volume of vehicles equipped with tire chains, and finally each was assumed to be R = 0.232 and R = 0.327.

Accordingly, in the case of wear depth researched at a rate of equipped tires at 10% of the total traffic volume, it was assumed that the use of the modified rate of vehicles equipped with tire chains was more effective in order to raise accuracy. It is necessary, in order to further raise accuracy, to measure the rate of vehicles equipped with tire chains more precisely.

1.3.2/ Prediction of Depth of Wear

In the examination of factors based on cumulative maximum depth of wear and traffic volume, Table 5 shows a list of correlation formulae whose correlation coefficients were 0.65 or more. It can be observed that the wear depth could be assumed by using winter period (X_1) and cumulative traffic volume of vehicles equipped with studded tires (X_7) or X_1 and cumulative traffic volume of vehicles equipped with studded tires (X_{19}) during the snowfall period.

Moreover, in a group of snowfall days, formula X_3 and cumulative total traffic volumes (X_4), X_3 and X_7, X_3 and the cumulative traffic volume of vehicles equipped with studded tires during the snowfall period (X_{15}) show high coefficients of correlation. Therefore, it may be possible to assume the wear depth by using two factors which can be measured most accurately among the values of X_1, X_3, X_4, X_7, X_{15}, X_{17}, etc.

Figure 11 shows the prediction of time when wear depth comes to 5, 10, and 15 mm using formula Number 11 which is the highest coefficient of correlation in Table 5. However, roads to which this Figure was applied were of road traffic section B or C which provide a snow removing rate of 25% to 100% and about 80% on average. Using this Figure, the depth of wear can be assumed by the cumulative traffic volume of vehicles equipped with studded tires during the snow removal period and snowfall period.

1.4/ Conclusion

As a result of research on the actual condition of road abrasion in regional highways, the following relationships could be confirmed.

1/ Wear depth of pavement during the winter period (about 120 days on average) is in the range of 2.5 mm to 20.0 mm maximum.

2/ The rate of vehicles equipped with studded tires is about 47% for the total traffic volume and about 28% for traffic volume of large vehicles. Japan's northern regions show a high equipped rate of studded tires.

3/ Factors concerning abrasion included the cumulative traffic volume of vehicles equipped with studded tires and cumulative total traffic volume during the winter period. However, cumulative traffic volume of vehicles equipped with tire chains can be said to have less effect than these two factors.

4/ Prediction formulae for wear depth will be of high accuracy if the cumulative traffic volume of vehicles equipped with studded tires during the snowfall period and snow removal period is used.

5/ Cumulative traffic volume and cumulative traffic volume of vehicles equipped with studded tires resulting in a wear depth of 10 mm are 40 x 10^4 vehicles and 14 x 10^4 vehicles, and the volumes resulting in a depth of wear of 20 mm are 280 x 10^4 vehicles and 90 x 10^4 vehicles.

6/ Fine-graded asphalt concrete and fine- and gap-graded asphalt concrete are widely used for asphalt pavement surfaces in snowy regions. For wear resistance, fine-graded asphalt concrete (13F) has the best performance.

2/ WEAR COMPARISON OF EACH TYPE OF MIXTURE

2.1/ Method of Research

Road wear due to tire chains and studded tires is very serious in snowy regions where there is danger of surface freezing. Therefore, wear-resistant pavements are needed in snowy regions.

For wear-resistant pavements, it is important to examine the materials and the mixing of materials; therefore, examination of various modified asphalts is now being conducted.

For modified asphalts, plastic modified asphalts mixed by thermosetting resins such as epoxy resin, etc. and thermoplastic resins such as rubberized asphalt mixed by styrene butadiene rubber (SBR), isoprene rubber (IR), chloroprene rubber (CR), etc., ethylene vinyl acrylate (EVA), ethylene ethyl acrylate (EEA), etc. are used. In typical snowy regions such as Hokkaido, Tohoku and Hokuriku, tests using a modified asphalt obtained by mixing rubberized asphalts have been carried out and these wear depths were researched in 1979 and 1980 and wear resistances were examined.

Wear depth in winter is set as the maximum wear depth (largest depth of concaves) and as the mean wear depth which is the average depth of concaves in the cross-section of the road, after measuring the sectional unevenness of roads at the beginning of the period when studded tires are installed (usually the end of November) and at the end of the period (usually the end of March). Moreover, as mentioned in Section 1, in the actual condition and prediction of wear, there are correlations between the maximum wear depth and mean wear depth; therefore, an analysis was made by using maximum wear depths because maximum wear depth appeared at questioned points of site.

2.2/ Results of Measuring

Based on wear depths in respective regions, Figures 12 to 23 show relationships between total traffic volume, traffic volume of heavy vehicles and traffic volume of vehicles equipped with studded tires in the winter period.

In Hokkaido, fine- and gap-graded asphalt concrete (F13) is mostly used as the mixture for wear resistant roads; second is dense-graded asphalt concrete (F13).

In the Tohoku region, dense- and gap-graded asphalt concrete (F13) is used most frequently followed with fine- and gap-graded asphalt concrete (F13) and fine-graded asphalt concrete (F13).

In the Hokuriku region, fine- and gap-graded asphalt concrete is used most frequently as specifications were provided in 1975.

The total traffic volume is largest in Hokkaido with the Hokuriku region next and and Tohoku the smallest. Moreover, in the Hokuriku region, the wear depth on rubberized asphalt sections is about 10% per million vehicles larger than for straight asphalt sections. This is because of differences of traffic volume between the two. Accordingly, wear depth in former asphalt sections is obviously larger, and cumulative traffic volumes during the winter period are $\bar{x} = 107 \times 10^4$ for former asphalt sections and $\bar{x} = 69 \times 10^4$ for latter asphalt sections.

In Tohoku, wear depths differ between straight asphalt and rubberized asphalt sections. Dense-graded asphalt concrete sections (F13) with rubberized asphalt are observed to have higher wear resistance. In Hokuriku region also, wear depth in rubberized asphalt concrete sections is smaller than in straight asphalt sections.

Wear depths at cumulative traffic volumes (N), 50×10^4, 100×10^4, and 300×10^4 are obtained by the traffic volume regression formula for large vehicles and are highly correlated. Table 6 shows the relationship.

From a comprehensive point of view, wear depth in sections which used rubberized asphalt concrete is better by about 10% to 20% compared with sections which use straight asphalt.

The relationship between traffic volume of heavy vehicles equipped with studded tires shows a low correlation, supposedly because those data were presumed to have traffic volume of vehicles equipped with studded tires the same as for other regions.

Research on the rate of vehicles equipped with studded tires was carried out in 1981. The research was made on Tuesdays of the second week from November, 1981 until May, 1982. Only data from November to March was used for data totalization as rates of vehicles equipped with studded tires in April and May are 0% regionally.

Figures 24 and 25 show the values $\bar{x}$ to 1σ of the rate of vehicles equipped with studded tires for large vehicles and small vehicles during the winter period. Rates of vehicles equipped with studded tires for both types of vehicles are high in Hokkaido, Iwate and Nagano.

Figures 26 to 28 show changes of the rates of vehicles equipped with studded tires for heavy vehicles in November, January and March. Accordingly, rates were observed to be low in November and to peak in January. The peak lasts until March.

3/ EXAMINATION ON TEST METHODS CONCERNING WEAR

3.1/ Ravelling Test

Ravelling generally means road surfaces of unsuitable asphalt mixture or concrete pavement being damaged by, not only tire chains, etc., but also by normal tire wear; however, recently it has come to mean abrasion due to studded tires as well.

An indoor ravelling test was made at the Hokkaido Development Bureau Civil Engineering Research Institute, in the first half of 1950 by rotating a wheel with chains fixed to it over a test sample. In 1954, a small size indoor testing machine was made on an experimental basis and used to test the sample by striking it with steel beads. In 1956, the ravelling testing was begun with a ravelling machine similar to the current asphalt pavement type ravelling machine. Even though this testing method indicated no function of rubbing chains as wheels came into contact with the test sample, the data from the test was gathered because of the urgent need for wear resistance data and because of the simple mechanism of that testing machine. Such a testing method is named "sample shuttling type" and from 1965 to 1975 various sample rotating-type test machines were developed in cold regions. A large revolving wear simulator equipped with actual truck tires was developed by the Japan Road Association.

Recently, respective regional construction departments and paving contractors concerned with cold regions have made some ravelling test machines. In the case of pavement wear resistance being estimated, the results are confirmed by an actual test on site. However, a ravelling test is necessarily made in order to determine the test method and to select the mixing design and materials before the actual site test.

3.2/ Joint Testing of Ravelling Test

As mentioned above, the ravelling test is a very popular means for determining the mix design of wear resistant mixtures. But, as various types of machines are provided in each organization and testing conditions are not unified, it is impossible to compare those test results directly. Besides, it is hard to standardize test machines and test methods because test data by test machine devices and test methods in each organization have already been accumulated. On the other hand, test machines tend to be changed to those for the studded-tire type because studded tires have become very popular. Tire chains are now only used for emergency measures. Regional differences do still exist, however. But for sometime yet, tire-chain, test machines will be used until a standard test machine for studded tires is completed.

Therefore, conversion methods of the respective data obtained by each test machine are examined.

3.2.1/ Ravelling Test Machine and Actual Condition of Test Method

Prior to starting the joint test, research on the actual condition of the ravelling test machine and test method was made in The Ministry of Construction and Hokkaido Development Bureau.

Consequently, test machines were provided for five organizations: the Hokkaido Development Bureau, the construction departments of Tohoku, Kanto, and Hokuriku regions, and the Public Works Research Institute. Table 1 shows the test machine dimensions, test machines and the test method. There are various test machines and test methods; however, there are similarities between test machines and test methods for the Construction Departments of Tohoku, Kanto and Hokuriku regions.

3.2.2/ Execution Method of Joint Test

A joint test carried out in 1979 was made by the former method; that of 1980 was made by a uniform method as far as possible. Test conditions are shown in Tables 7 and 8.

For asphalt concrete samples, aggregate from Hokkaido (Table 9) is used and mixing is shown in Table 10. Sample forming is shown in Table 11 and, in the 1979 test, materials from Hokkaido were sent directly to each organization and unified samples were formed. In the 1980 test, samples were formed in Hokkaido and then sent to the respective organizations.

3.2.3/ Joint Test Result in Former Methods of Each Organization

The 1979 Test Result in shown in Table 12 and Figure 29.

Abrasion volumes are different largely because test temperature, test times, wear action and sample forming, etc. were not the same for each organization.

Abrasion volumes by the rotating-type test machine are divided into two groups: the larger one appeared in the Hokkaido Development Agency and Public Works Research Institute where cross chains were used and the smaller one appeared in the Construction Departments of Tohoku, Kanto and Hokuriku regions where side chains were used. The proportion is about 3:1. In addition, the abrasion volume in the shuttling-type test machine with different chains carried out in the Hokkaido Development Bureau is about twice as large for cross chains as for side chains. Accordingly, the quality of chain is supposed to affect abrasion greatly even though there are many other factors.

Degree of compaction is 99.5% to 102.0% for the four organizations except for the Hokkaido Development Bureau with 93.2% to 94.6%. The reason may be that these four organizations used roller compacters as compacting machines, while the Hokkaido Development Bureau used a compression test machine. The asphalt pavement outline prescribes the degree of compaction of samples for indoor wheel-tracking to be 100 ± 1%. Compared with this degree of compaction, the sample in the Hokkaido Development Bureau was assumed to have an insufficient degree of compaction so the abrasion was large. Accordingly, in forming mixtures containing crushed stone, it is desirable to use roller compacters which provide sufficient compaction.

Abrasion volume $\sqrt{v}$ in the same organization was 0.3 and less than 0.2 in the others. But all coefficients of variation were less than 20% and so dispersion was small.

3.2.4/ Joint Test Results in Uniform Test Conditions

As the dispersion of results of the 1979 test was very wide due to test conditions, abrasion volume in the 1980 test was compared by unifying test conditions. The test results are shown in Table 13 and Figure 30.

The abrasion volume in the same test time is different in each organization. As observed in Table 8, the reasons are that the revolution velocity (wheel and test sample) and the number of wheels are different. Particularly in the case of the Hokkaido Development Bureau, abrasion volume is twice as much as that of other organizations because the number of wheels is twice that of other organizations, but the number of chains is less than two per wheel. In the case of the Public Works Research Institute, the abrasion volume is similar to that of other organizations because the test conditions are almost the same, and the wheel diameter is about twice and velocity about half though the number of chains per wheel is about twice as much as that of other organizations. Therefore, by making the value for the rotating-type machine at the construction department of Hokkaido region one-half, abrasion volumes during the same test times (three hours) by the rotating-type machine are 0.74 to 1.57 for fine-graded (OAC) asphalt volume, 0.38 to 1.30 for dense-graded and 0.98 to 2.74 for fine-graded (OAC — 0.8% and 1%) asphalt volume and 0.82 to 2.44 for dense-graded. This dispersion is assumed to be affected by velocity of rotation. Test values of the organiztions cannot be directly compared because the different rotating velocity cannot be easily modified on their mechanisms.

3.2.5/ Conversion of Test Values Among Organizations

It is impossible to compare test values accurately as test equipment is different, but as a rough standard, test values between organizations were compared.

First, from the 1979 test result, relationships between abrasion volumes by former test methods carried out in each organization were compared and the results are shown in Table 14. The standard is the prescribed shuttling-type method with side chains for asphalt .

Next, abrasion volumes during a three-hour test were compared by standardizing the shuttling-type with the cross-chain test values from Hokkaido, and the results are shown in Figure 15. Moreover, test times to obtain the abrasion volume carried out at the Hokkaido Development Bureau are shown in Figure 15.

Also, test time to obtain equal abrasion volume with abrasion volume by shuttling-type test machines during one hour and a half is shown in Table 16. Accordingly, dispersion of test times is very wide with a minimum three hours and maximum 18 hours with test time for the Construction Department of Tohoku region the largest at 18 hours.

3.3/ Future Problems

1/ Test Machine

As mentioned in Sections 1 and 2, if a tire-chain-type test machine is used, the wear mechanism is different from the actual road wear mechanism, because most vehicles are equipped with studded tires, with tire chains being used only for emergencies in certain areas. Therefore, the purpose of simulation tests is defeated if wear resistance of mixtures is estimated by only the tire-chain type. Therefore, examination of determining standard test machines and unified test conditions will be made.

However, a studded-tire-type, small test machine is still under development. The mechanism and test method of the tire-chain type is comparatively simple. Therefore, in conclusion, it is proper to extend interchangeability while using the present test machines.

On the other hand, organizations providing the studded-tire-type, test machine have decided to confirm the correlation between the two types and problems of chain type, and to prepare data to determine future standard test methods.

2/ Relationship Between the Abrasion of Actual Roads and Wear Resistant Mixtures

In mixing design mixtures, a standard value as an index of wear resistance is necessary. Therefore, it is necessary to fix the life of the wear resistant mixture around three or six years. There are different problems to consider, such as aggregate circumstances and traffic condition, etc. for respective regions.

Accordingly, it is necessary to grasp the relationship between actual wear conditions of respective regions and the indoor appraisal of used mixtures.

3/ Tire Chain

If present test machines are to be used for some time, tire chains which can be immediately used in this test method should be considered.

For fine-graded mixtures such as asphalt mortar, the test method described in the asphalt pavement outline (Chain: G4051 S35C) is suitable, and for direct testing of present mixtures formed mainly from crushed stone, it will be an accelerated test if a cross chain (JIS D4241-B-5, quality JIS G3505, SWRM3) is used.

4/ Improvement Measures to Decrease Dispersion of Test Samples

Compaction of test samples shall be made by the amsler universal test machine according to the test method described in the asphalt pavement outline. In the case of forming mixtures containing a high percentage of crushed stone, a roller compactor is preferably used because static loads cannot give enough compaction to the test sample.

5/ Others

As abrasion volumes differ according to quality, weight and velocity of chains and with the temperature condition, it is necessary to examine abrasion volume under these conditions from now on.

Regarding information exchange, it is suggested that test conditions shall be attached to data in order to convert test values.

3.4/ Conclusion

Matters agreed upon in this joint test are as follows.

1/ Rotating-type test equipment is being widely used as a ravelling test machine, and there are various wear mechanisms and test methods. The Construction Department of Tohoku, Hokuriku and Kanto regions are using similar test machines.

2/ There was dispersion among test results by the former test methods of various organizations, with maximum abrasion volume being five to six times the minimum volume.

Table 14 is used to convert abrasion volumes by the various organizations to abrasion volume by shuttling type.

3/ In test results by unified test conditions such as test sample, chain etc., the maximum abrasion volume among respective organizations is 3.5 times as much as the minimum volume. This is caused by differences of rotating velocities of wheels and test samples, but it is recognized that unifying the velocities will be difficult.

The coefficient of conversion to the same time is shown in Table 15 and test time to obtain abrasion volumes corresponding to the volume of going and returning type in one hour and a half is shown in Table 16.

4/ For some time, the present test machines will be used.

5/ There is a relationship between the actual condition of road wear and used mixtures in respective organizations.

6/ The test of mixtures containing crushed stone will be an accelerated test, if cross chains are used. Roller compactors are better for compaction of test samples.

Table 1 Measuring Data (1977)

			Research period (days)	Temperature			Snow removal period (days)	Snow car distinguishing edge use period (days)	Freezing index	Number of traffic lanes	Autumn (12 hours)		Winter (12 hours)		Autumn (24 hours)		Winter (24 hours)		Entire traffic volume in types of tires				Traffic volume of heavy vehicles in types of tires				Depth of wear (mm)	
				Minimum	Maximum	Mean					Entire traffic volume	Traffic volume of heavy vehicle	Entire traffic volume	Traffic volume of heavy vehicle	Entire traffic volume	Traffic volume of heavy vehicle	Entire traffic volume	Traffic volume of heavy vehicle	S M	S P	N O	T C	S M	S P	N O	T C	Maximum	Mean
1	Hokkaido	Regional (Do) highway	122	-15.7	5.5	-4.9	50		461.0	2	8,107	691	6,409	757	16,654	2,236	13,630	1,800	25.6	74.4	0	0	65.0	34.3	0	0.7	4.0	1.6
2		Regional (Do) highway	122	-15.7	5.5	-4.9	50		461.0	2	8,107	691	6,409	757	16,054	2,236	13,630	1,800	25.6	74.4	0	0	65.0	34.3	0	0.7	5.0	2.0
3	Aomori	National highway Route 101	121	-12.0	14.5	-0.9	45.5		123.7	1	8,230	559	6,062	342	16,887	1,229	12,428	754	25.0	50.0	0	18.0	25.0	50.0	0	16.0	12.0	5.3
4		Regional (Ken) highway	122	-12.3	13.5	-1.5	61.0		176.2	2	10,740	961	11,403	887	21,096	1,921	23,218	3,506	22.9	71.0	0	6.1	22.9	71.0	0	6.1	14.0	9.3
5		Regional (Ken) highway	123	-17.9	13.2	-2.1	60		205.1	1	7,130	635	5,700	615	14,615	1,360	12,665	1,280	10.0	85.0	2.0	3.0	22.0	65.0	2.5	10.5	13.0	6.1
6		Regional (Ken) highway	122	-15.0	16.3	-1.2	73		193.1	2	11,384	1,622	5,071	651	22,950	3,096	10,488	1,364	9.6	90.4	0	0	75.4	24.6	0	0	3.0	1.1
7		Regional (Ken) highway	122	-12.8	12.2	-2.2	66		214.7	1	4,029	778	2,763	557			6,076	1,120	13.0	75.0	0	12.0	13.0	75.0	0	12.0	9.0	3.3
8	Miyagi	Regional (Ken) highway	100	-9.0	17.4	3.6	66.5		28.0	1	604	60	417	29	1,377	125	961	58	34.8	54.0	7.6	3.6	40.9	45.5	6.8	6.8	4.0	0.6
9		Regional (Ken) highway	122	-14.0	10.0	-1.6	80.3		151.6	1	12,002	2,630	11,726	1,314	24,323	9,578	23,978	3,154	11.3	79.8	2.2	6.7	58.7	22.9	1.3	17.1	8.0	1.5
10	Yamagata	National highway Route 287	93	-20.0	15.0	-1.5	86.5	27.0	153.2	1	4,102	340	6,464	540	8,388	838	13,103	1,146	71.5	21.6	2.0	4.9	25.0	9.9	5.0	60.1	9.3	6.8
11		Regional (Ken) highway	130	-11.9	16.4	-2.5	82.0		157.0	2	10,610	2,252	8,480	2,139	21,537	5,413	17,238	5,142	2.3	78.1	0	16.6	92.0	0	0	8.0	2.5	1.2
12	Fukushima	Regional (Ken) highway	104	-12.0	4.0	-1.4	72.0	18.0	171.0	1	3,039	438	7,061	1,294	6,161	961	16,228	2,588	62.9	35.5	0.7	0.9	62.9	35.5	0.7	0.9	6.0	2.8
13		National highway Route 121	122	-7.0	21.0	2.0			5.6	2	16,102	410	11,648	320	32,543	1,636	23,514	663	64.7	18.3	0	17.0	43.7	7.4	0	48.9	4.0	1.6
14	Tochigi	National highway Route 120	122	-15.0	10.6	-4.0	82.0		150.9	1	16,176	1,438	10,766	1,245	29,284	4,280	22,261	3,250	10.0	12.0	43.0	35.0	38.0	8.0	49.0	5.0	1.0	1.2
15		Regional (Ken) highway	124	-10.0	15.1	2.0	81		0	2	6,464	489	1,380	172	9,078	1,130	6,946	894	37.0	14.0	39.0	10.0	13.9	13.9	18.9	8.3	8.0	2.7
16	Gunma	National highway Route 144	133	-15.0	20.0	-2.9	21		256.4	1	1,521	483	1,290	262	1,427	1,330	2,843	743	43.1	17.9	12.3	26.5	53.0	3.7	15.4	67.0	3.0	1.2
17		Regional (Ken) highway	121	-10.0	15.0	-1.4	61		118.0	1	4,141	392	4,057	271	8,078	976	6,235	661	58.9	11.1	21.0	7.0	38.9	0.6	32.1	15.3	2.5	1.0
18	Niigata	Regional (Ken) highway	122	-11.0	14.0	2.0	75.0	35.0	0	3	7,272	920	17,083	1,664					98.9	0	0	1.1					10.0	2.0
19		Regional (Ken) highway	120	-10.6	17.2	2.9	90		0	2	802	31	644	29	1,615	73	1,318	60	35.9	23.4	0	40.7	35.9	23.4	0	40.7	9.0	2.7
20	Nagano	National highway Route 18	122	-7.0	9.7	0.4	90		42.0	2	7,670	3083	4,631	1,265	19,366	11,790	11,628	4,332	12.0	80.1	6.6	1.3	9.0	78.6	10.5	1.9	3.0	1.0
21		National highway Route 147	118	-9.5	3.6	-3.0	86.5		261.6	2	4,212	1,193	3,052	995	10,864	6,607	7,685	3,274	12.8	80.0	5.0	1.6	11.0	80.0	7.0	2.0	7.0	3.6
22	Toyama	Regional (Ken) highway	116	-13.0	28.0	5.1	72.0		0	2	16,009	1,236	1,070	135	33,836	4,250	2,288	334	47.7	18.8	0	33.5	76.9	0	0	23.1	17.0	8.0
23		Regional (Ken) highway	122	-3.5	10.1	3.5	77		0	2	10,855	1,320	11,695	776	28,106	2,620	27,670	1,648	38.9	60.0	0	0.1	95.3	0	2.4	2.3	10.0	4.4
24	Gifu	National highway Route 156	118	-16.7	9.6	-2.9	69		206.0	1	4,129	478	2,980	207	8,140	1,026	5,211	610	40.0	20.0	10.0	30.0	60.0	20.0	10.0	10.0	1.0	0.4
25		National highway Route 156	121	-11.0	12.0	2.2	70.5	23.0	47.6	1	2,921	431	3,630	246			7,361	493	58.7	5.0	35.3	0	78.5	0.4	21.1	0	7.5	1.9
26	Fukui	National highway Route 158	152	-7.1	18.5	3.5	70		0	2	17,202	963	12,163	116			24,126	232	75.0	25.0	0	0	72.0	28.0	0	0	4.6	1.0
27		Regional (Ken) highway	152	-7.1	18.5	3.5	70		0	1	8,743	930	2,481	708			4,886	216	75.0	25.0	0	0	75.0	25.0	0	0	6.0	1.0
28	Shimane	Regional (Ken) highway	126	-4.2	18.6	6.0	64		0	2	7,380	1,111	6,752	1,099	14,758	3,386	13,648	2,348	30.0	20.0	20.0	30.0	48.0	1.6	30.0	20.0	4.0	1.0

Table 2 Result of Research on Actual Condition of Wear

Items		Character	Number of research points (n)	Maximum value (max)	Minimum value (min)	Mean value ($\bar{x}$)	Square root of unbiased variance ($\sqrt{v}$)
Research period (days)			82	191.0	75.0	120.34	0.134
Temperature (°C)		Minimum	74	0.9	-20.0	-9.41	-0.490
		Maximum	74	27.5	2.3	14.59	0.368
		Mean	74	9.2	-4.9	1.29	2.482
Snow removal days (days)			70	90.0	23.0	72.38	0.207
Snow extinguishing pipe use days (days)			10	38.0	7.0	24.60	0.352
Freezing index (°C·days)			74	461.0	0.0	83.19	1.307
Number of traffic lanes			84	3.0	1.0	1.54	0.368
Traffic volume (vehicles/days/direction)	Autumn period	Entire traffic volume	77	14,136.0	401.0	4,917.42	0.600
		Traffic volume of heavy vehicles	77	2,630.0	15.5	648.18	0.784
	Winter period	Entire traffic volume	78	12,068.0	322.0	4,062.26	0.634
		Traffic volume of heavy vehicles	78	2,752.0	14.5	519.06	0.974
Number of passed wheels (wheels/days/direction)	Autumn period	Entire traffic volume	69	29,284.0	765.5	9,866.88	0.615
		Traffic volume of heavy vehicles	69	6,310.0	36.5	1,596.50	0.828
	Winter period	Entire traffic volume	73	25,196.0	659.0	8,486.97	0.627
		Traffic volume of heavy vehicles	73	6,547.0	34.0	1,273.28	0.962
Structure proportion in types of tires (%)	Entire traffic volume	SN	71	98.9	0.0	39.19	0.635
		SP	71	95.1	0.0	46.69	0.640
		NO	71	48.1	0.0	7.29	1.611
		TC	71	57.5	0.0	7.44	1.554
	Traffic volume of large vehicles	SN	69	100.0	0.0	52.69	0.526
		SP	69	93.4	0.0	28.18	1.004
		NO	69	52.5	0.0	9.27	1.552
		TC	69	67.0	0.0	9.98	1.589
Depth of wear (mm)		Maximum	80	20.0	1.0	6.67	0.580
		Mean	80	11.9	0.4	2.54	0.793

Table 3 Depth of Wear in Types on Surface Mixture

Types of asphalt concrete / Item		Fine graded	Fine graded (F13)	Fine and gap graded	Dense graded	Asphalt mortar As	Mastic asphalt As
Number of research points		10	6	5	4	2	1
Number of data		30	18	13	10	6	3
Maximum depth of wear (mm)	max	18.0	13.0	17.0	9.0	8.0	20.0
	min	1.0	3.0	2.0	4.0	3.0	2.5
	$\bar{x}$	7.27	5.39	7.54	5.90	4.83	10.83
	σ	4.605	2.544	3.479	1.853	1.772	8.780
Mean wear depth (mm)	max	11.9	6.1	8.0	3.5	2.7	3.7
	min	0.4	0.6	1.0	0.9	0.9	1.0
	$\bar{x}$	3.05	1.98	3.13	1.75	1.68	2.60
	σ	2.696	1.403	1.744	0.920	0.833	1.418

Table 4 Coefficient of Correlation (1979)

	Winter period (days)	Snow removal period (days)	Snowfall period (days)	Converted traffic volume		Cumulative traffic volume (Σ log)																			
						Winter period								Snow removal period (days)				Snowfall period (days)							
						Converted traffic volume of small vehicles				Converted traffic volume of heavy vehicles				Small vehicles conversion		Heavy vehicles conversion		Converted traffic volume of small vehicles				Converted traffic volume of heavy vehicles			
				Small vehicles conversion	Heavy vehicles conversion	Entire traffic volume	Studded tire	100-(studded tire + snow tire)	100-studded tire	Entire traffic volume	Studded tire	100-(studded tire + snow tire)	100-studded tire	Entire traffic volume	Studded tire	Entire traffic volume	Studded tire	Entire traffic volume	Studded tire	100-(studded tire + snow tire)	100-studded tire	Entire traffic volume	Studded tire	100-(studded tire + snow tire)	100-studded tire
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25
1	0.000	0.521	0.518	0.492	0.467	0.491	0.626	0.475	0.454	0.466	0.550	0.491	0.464	0.461	0.575	0.455	0.528	0.526	0.625	0.461	0.450	0.483	0.559	0.475	0.452
2		0.000	0.511	0.392	0.335	0.398	0.602	0.252	0.241	0.140	0.461	0.276	0.284	[illegible]	0.538	0.288	0.424	0.476	0.609	0.242	0.279	0.402	0.493	0.282	0.259
3			0.000	0.505	0.537	0.574	0.656	0.515	0.509	0.530	0.592	0.527	0.500	[illegible]	0.641	0.541	0.594	0.549	0.617	0.528	0.500	0.509	0.572	0.556	0.505
4				0.000	0.399	0.399	0.608	0.488	0.579	0.395	0.488	0.509	0.635	0.498	0.581	0.461	0.460	0.465	0.593	0.417	0.412	0.412	0.511	0.439	0.434
5					0.000	0.412	0.647	0.360	0.350	0.321	0.445	0.443	0.527	[illegible]	0.562	0.356	0.416	0.467	0.595	0.326	0.319	0.365	0.463	0.379	0.398
6						0.000	0.610	0.516	0.575	0.408	0.483	0.530	*0.658	0.451	0.518	0.467	0.461	0.483	0.591	0.448	0.434	0.409	0.507	0.466	0.469
7							0.000	0.617	0.608	0.649	0.608	0.636	0.652	[illegible]	0.617	0.454	0.609	0.602	[illegible]	[illegible]	[illegible]	0.419	0.602	0.627	0.637
8								*0.000	0.156	0.373	0.461	0.118	0.099	[illegible]	[illegible]	[illegible]	0.406	0.614	[illegible]	[illegible]	[illegible]	0.417	0.457	0.011	0.225
9									0.000	0.355	0.456	0.192	0.117	[illegible]	[illegible]	[illegible]	0.414	0.511	[illegible]	[illegible]	0.238	[illegible]	0.473	0.120	0.156
10										0.000	0.445	0.461	0.524	[illegible]	0.557	0.345	0.417	0.471	[illegible]	[illegible]	0.422	[illegible]	0.461	0.402	0.382
11											0.000	0.400	0.441	0.459	0.514	0.441	0.462	0.524	[illegible]	0.441	0.471	0.459	0.456	0.444	0.442
12												0.000	0.179	0.367	0.567	[illegible]	0.417	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0.463	0.138	0.317
13													0.000	[illegible]	[illegible]	0.411	0.411	[illegible]	[illegible]	0.464	[illegible]	0.425	0.474	0.102	0.189
14														[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0.488	0.124	0.317
15															[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0.508	0.509
16																[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0.297	0.271
17																	[illegible]	[illegible]	[illegible]	0.413	0.440	0.444	[illegible]	0.409	0.412
18																		[illegible]	[illegible]	[illegible]	[illegible]	0.491	[illegible]	0.439	0.672
19																			[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0.443	0.637
20																				[illegible]	[illegible]	0.439	0.457	0.138	0.180
21																					[illegible]	0.371	0.476	0.390	0.213
22																						[illegible]	0.412	0.549	0.518
23																							[illegible]	0.466	0.456
24																								0.000	0.288
25																									0.000
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25
Maximum depth of wear	*0.621	0.605	0.600	0.465	0.361	0.409	0.600	0.151	0.231	0.365	0.291	0.063	0.109	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0.167	0.187	0.325	0.199	0.315	0.108	0.260
Mean depth of wear	0.490	0.241	0.500	0.390	0.319	0.394	*0.599	0.005	0.111	0.321	0.001	-0.052	0.080	[illegible]	[illegible]	0.266	[illegible]	[illegible]	0.568	0.063	0.205	0.362	0.456	-0.001	0.155

N = 39

* : Maximum value

Formula of multiple correlation $Y = A0 + A1 \cdot X1 + A2 \cdot X2$

Formula of singular correlation $Y = A0 + A1 \cdot X1$

Table 5 Prediction Formulae of Wear Depth

1) Maximum wear depth

No.	$Y = A0 + A1 \cdot Xi + A2 \cdot Xj$	R
1	$Y = -18.770 + 0.051 \times X1 + 5.426 \times X4$	0.651
2	$Y = -40.566 + 0.041 \times X1 + 8.194 \times X7$	0.713
3	$Y = -31.158 + 0.044 \times X1 + 6.679 \times X15$	0.688
4	$Y = -33.071 + 0.046 \times X1 + 6.572 \times X18$	0.673
5	$Y = -30.493 + 0.044 \times X1 + 6.774 \times X19$	0.712
6	$Y = -40.146 + 0.047 \times X2 + 9.468 \times X19$	0.669
7	$Y = -30.690 + 0.092 \times X3 + 9.247 \times X4$	0.707
8	$Y = -45.050 + 0.089 \times X3 + 8.492 \times X6$	0.693
9	$Y = -40.762 + 0.071 \times X3 + 8.718 \times X7$	0.718
10	$Y = -40.006 + 0.100 \times X3 + 7.794 \times X14$	0.706
11	$Y = -37.915 + 0.082 \times X3 + 8.402 \times X15$	0.731
12	$Y = -19.670 + 0.098 \times X3 + 5.022 \times X16$	0 658
13	$Y = -29.327 + 0.076 \times X3 + 6.488 \times X18$	0.655
14	$Y = -23.549 + 0.067 \times X3 + 6.114 \times X19$	0.668
15	$Y = -71.681 + 20.843 \times X7 - 6.548 \times X11$	0.686
16	$Y = -72.785 + 20.609 \times X7 - 6.375 \times X17$	0.689
17	$Y = -71.336 + 19.053 \times X7 - 4.836 \times X23$	0.662
18	$Y = -69.962 + 27.341 \times X18 - 15.283 \times X25$	0.671
19	$Y = -51.081 + 17.387 \times X19 - 5.816 \times X23$	0.653

2) Mean wear depth

No.	Y = A0 + A1 · Xi + A2 · Xj	R
1	$Y = -26.454 + 0.028 \times X3 + 5.418 \times X7$	0.656
2	$Y = -45.518 + 17.992 \times X6 - 11.456 \times X13$	0.658
3	$Y = -25.503 + 8.538 \times X7 - 3.179 \times X13$	0.652
4	$Y = -29.083 + 10.080 \times X7 - 4.214 \times X16$	0.654

Y : Depth of wear (millimeter)
X1 : Winter period (days)
X2 : Snow removal period (days)
X3 : Snowfall period (days)
X4 : Entire traffic volume (log vehicles/days/one direction)
X6 : Cumulative traffic volume during winter period (Σlog vehicles/days/one direction)
X7 : Cumulative traffic volume of vehicles equipped with studded tires during winter period (Σlog vehicles/days/one direction)
X11: Cumulative traffic volume of heavy vehicles equipped with studded tires during winter period (Σlog vehicles/days/one direction)
X13: Cumulative modified traffic volume of heavy vehicles equipped with tire chains during winter period (Σlog vehicles/days/one direction)
X14: Cumulative traffic volume during snow removal period (Σlog vehicles/days/one direction)
X15: Cumulative traffic volume of vehicles equipped with studded tires during snow removal period (Σlog vehicles/days/one direction)
X16: Cumulative traffic volume of heavy vehicle tires during snow removal period (Σlog vehicles/days/one direction)
X17: Cumulative traffic volume of heavy vehicles equipped with studded tires during snow removal period (Σlog vehicles/days/one direction)
X18: Cumulative traffic volume during snowfall period (Σlog vehicles/days/one direction)
X19: Cumulative traffic volume of vehicles equipped with studded tires (Σlog vehciles/days/one direction)
X23: Cumulative traffic volume of heavy vehicles equipped with studded tires (Σlog vehicles/days/one direction)
X25: Cumulative modified traffic volume of heavy vehicles equipped with tire chains (Σlog vehicles/days/one direction)
R : Coefficient of correlation

Table 6

Regions	$N \times 10^4$	① Wear depth of straight asphalt section (mm)	② Wear depth of rubberized asphalt section (mm)	②/① (%)
Hokkaido region	50	5.5	4.4	0.80
	100	6.7	7.6	1.13
	300	12.0	18.3	1.53
Tohoku region	50	4.2	4.2	1.00
	100	6.3	5.8	0.92
	300	13.5	11.2	0.83
Tohoku region	50	4.2	3.7	0.88
	100	7.1	6.4	0.90
	300	16.6	16.0	0.96
Hokuriku region	50	6.3	5.2	0.83
	100	8.0	6.9	0.86
	300	13.3	12.4	0.93

Table 7 Dimensions of Ravelling Test Machine and Test Method (Test Conditions in 1979)

Items \ Name of organization			Hokkaido Development Bureau		Construction department of Tohoku region	Construction department of Kanto region	Construction department of Hokuriku region	Public works research institute	
Dimensions of test machine	Type		Rotating type	Shuttling type	Rotating type	Rotating type	Rotating type	Rotating type	
	Moving (by wheel or test sample)		Test sample	Test sample	Test sample	Test sample	Test sample	Test sample	
	Outer diameter and width of wheel		270,100 (mm)	250,100 (mm)	270,100 (mm)	250,100 (mm)	250,100 (mm)	Steel wheel, Tire for small car 500,100(mm) (5.0-10-4P)	
	Number of wheels		Two wheels	One wheel	One wheel	One wheel	One wheel	One wheel	
	Size of table (diameter of running road)		150 cm	——	110 cm	100 cm	110 cm	100 cm	
	Size of test sample		40 × 15 × 5(cm)	40 × 15 × 5(cm)	$\frac{33}{55}$ × 25(cm)	$\frac{30}{50}$ × 25(cm)	$\frac{32.8}{53.5}$ × 25 × 5(cm)	$\frac{33}{50}$ × 25 × 5(cm)	
Test conditions	Wear action		Tire chain (cross chain)	Chain	Chain	Chain	Chain	Chain	Tire chain for small car
	Table velocity		16 revolutions per minute	66 shuttling times per minute	20 revolutions per minute	5 revolutions per minute	2 revolutions per minute	26.5 ~ 132.7 revolutions per min. 5.6 km/h	10 km/h
	Number of wheel rotations		212 rotations per minute	200 rotations per minute	200 rotations per minute	200 rotations per minute	200 rotations per minute	111 ~ 555 revolutions per minute 11.8 km/h	11 km/h
	Wheel load		Zero kilogram (void)	0 kg	0 kg	0 kg	0 kg	0 ~ 500 kg 0 kg (void)	1.5 kg/cm²
	Test temperature		-10°C ± 1	-10°C ± 1	-10°C ± 1	0°C ± 1	0°C ± 1	-10° C ± 1	Sprinkling 10°C ± 1
	Test time		One side 3 hr × 2	One side 1.5 hr × 2	About 10 hours	3 hours	5 hours	3 hours	6 hours
	Chain	Materials	JIS D4201 B-5 [illegible]) (cross ring)	JIS G4051 S35C	JIS G4051 S35C	JIS G4051 S35C	JIS G4051 S35C	JIS G4051 S35C	Tire chain for small car (cross ring)
		Number	10 frames × 10 pieces × 2	11 frames × 12 pieces × 1	11 frames × 12 pieces × 1	10 frames × 12 pieces × 1	6 frames × 12 pieces × 1	10 frames × 24 pieces × 1	1 set × 1
	Measure of wear depth		Average value of inside and outside of replacement method	Average value of inside and outside 6 sections of dial gauge method	Iron sand replacement method dial gauge method	Average value of 3 sections of dial gauge method	Average value of 3 sections of dial gauge method	Average value of 2 sections of dial gauge method	

Table 8 Test Conditions 1980

Name of organization / Items	Hokkaido		Tohoku	Kanto	Hokuriku	PWRI
	Rotating type	Shuttling type				
Test temperature	-10 ± 1°C	-10 ± 1°C	-10°C ± 1°C	-10°C ± 1°C	-10°C ± 1°C	-10°C ± 2°C
Curing time	24 hours or more	24 hours or more	24 hours or more	24 hours or more	24 hours or more	17 hours or more
Chain material	Cross chain JIS G3505 SWRM3	ditto	ditto	ditto	ditto	ditto
Number of chains and wheels	10 frames × 10 pcs × 2 wheels	10 frames × 12 pcs × 1 wheel	10 frames × 12 pcs × 1 wheel	10 frames × 12 pcs × 1 wheel	10 frames × 12 pcs × 1 wheel	10 frames × 24 pcs × 1 wheel
Number of wheel rotations	212 revolutions per minute	200 revolutions per minute	200 revolutions per minute	200 revolutions per minute	200 revolutions per minute	125.4 ± 3.3 rotations per minute
Number of test sample rotations	16 rotations per minute	66 shuttling times per minute	19.2 rotations per minute	5 rotations per minute	2 rotations per minute	29.9 ± 0.5 rotations per minute
Distance between bolt for installing chain and surface of test sample	10 cm	10 cm	10 cm	10 cm	10 cm	10 cm
Measuring method of wear depth	Dial gauge 3 sections	ditto	ditto	ditto	ditto	dial gauge 2 sections

Table 9 Characteristics of Used Material

Items / Materials	Percentages of passed weight (%)							Specific gravity (g/cm^3)	Amount of water absorption (%)	Amount of loss (%)
	13mm	5	2.5	0.6	0.3	0.15	0.074			
Crushed stone No. 6	100	0	-	-	-	-	-	2.600*)	1.60	-
Crushed stone No. 7	-	100	0	-	-	-	-	2.600*)	1.60	-
Coarse sand	-	100	94	57	26	5	0	2.714*)	1.25	1.2
Fine sand	-	-	-	100	98	12	2	2.616*)	1.82	0.5
Stone dust	-	-	-	-	100	97	88	2.709*)	-	-

*) Specific gravity of saturated surface

Table 10 Result of Mix Design

Types of mixtures		Fine and gap graded asphalt concrete		Dense and gap graded asphalt concrete	
Mix ratio (%)	Amount of binder	6.0	7.0	5.0	5.8
	Crushed stone No. 6	25.1	24.3	43.3	42.2
	Crushed stone No. 7	14.2	13.8	14.7	14.3
	Coarse sand	26.7	25.8	13.8	13.5
	Fine sand	17.8	17.2	14.7	14.3
	Stone dust	10.2	11.9	8.5	9.9
Combined gradation (%)	13mm	100		100	
	5	74		55	
	2.5	57		39	
	0.6	47		34	
	0.3	38		29	
	0.15	16		13	
	0.074	12		10	

Table 11 Producing Conditions of Test Sample

Item	Execution organization / Year of execution	1979					1980
		Hokkaido	Tohoku	Kanto	Hokuriku	PWRI	Hokkaido
Asphalt heating	Temperature (°C)	145	145	140	140	140 ∿ 150	150
	Time (minute)	15	20	60	10	20	15
Aggregate heating	Temperature (°C)	175	150	140	150	140 ∿ 150	175
	Time (hr)	4	4 hours or more	19	16	12	4
Mixing	Temperature (°C)	145	145	140	150	140	150
	Time (sec)	180	120	Dry 60 Wet 120	Dry 60 Wet 120	Dry about 60 Wet 180	180
	Mixer capacity	30 kg	45 kg	20 ℓ	20 ℓ	60 kg	30 kg
	Number of rotations	Uni-axial 60rpm Bi-axial 20rpm	Revolution 10rpm Rotation 35rpm	Uni-axial 64rpm Bi-axial 23rpm	23rpm	45rpm	Uni-axial 60rpm Bi-axial 20rpm
Compaction	Temperature (°C)	135	135	110	120	110	139
	Machine types	Compression testing machine	Roller compactor	ditto	ditto	ditto	ditto
	Load or linear compression	20 t	62.2 kg/cm	13.3 kg/cm, 31.7 kg/cm	40 kg/cm	60 kg/cm	20 kg/cm
	Time or times	5 minute	7 shuttling times	10 shuttling times, 15 shuttling times	Steel wheel 5 shuttling times Rubber plate 5 shuttling times	20 shuttling times	Steel and rubber 22 shuttling times steel 5 shuttling times

Table 12 Test Result 1979

Items	Mixtures / Test organization	Fine and gap graded asphalt concrete							Dense and gap graded asphalt concrete						
		*2) Tohoku	*2) Kanto	*2) Hokuriku	*1) PWRI	Hokkaido			*2) Tohoku	*2) Kanto	*2) Hokuriku	*1) PWRI	Hokkaido		
						*1) Rotating type	*2) Shuttling type	*1) Shuttling type					*1) Rotating type	*2) Shuttling type	*1) Shuttling type
n		3	3	3	3	3	2	2	3	3	3	3	3	2	2
Depth of wear (cm^2)	$\bar{x}$	0.80	0.81	0.86	2.90	2.77	1.17	2.40	0.35	0.89	0.76	2.65	3.00	1.58	2.87
	$\sqrt{V}$	0.13	0.005	0.14	0.30	0.10	0.09	0.18	0.06	0.06	0.03	0.15	0.14	0.10	0.22
	S	16.3	0.6	16.3	10.3	3.6	7.7	7.5	17.1	6.7	3.9	5.7	4.7	6.3	7.7
Density (g/cm^3)	$\bar{x}$	2.339	2.315	2.378	2.353	2.207	2.187	2.192	2.387	2.364	2.369	2.352	2.212	2.184	2.196
	$\sqrt{V}$	0.005	0.007	0.007	0.007	0.003	0.004	0	0.002	0.004	0.007	0.006	0.01	0.006	0.004
	δ	0.2	0.3	0.3	0.3	0.1	0.2	0	0.1	0.2	0.3	0.3	0.5	0.3	0.2
Compaction degree (%)	$\bar{x}$	99.7	99.5	101.3	100.3	94.0	93.2	93.4	102.0	101.0	101.3	100.5	94.6	93.3	93.9
	$\sqrt{V}$	0.21	0.32	0.26	0.30	0.12	0.14	0	0.12	0.15	0.32	0.26	0.57	0.20	0.21
	δ	0.2	0.3	0.3	0.3	0.1	0.2	0	0.1	0.1	0.3	0.3	0.6	0.2	0.2

*1) Cross chain

*2) Side chain

Table 13 Test Result 1980

Mixtures		Test organization		Number of test sample × Measured sectional area	Density $\bar{x}_2$ (g/cm^3)	Depth of wear $\bar{A}$ (cm^2) (per hour)						
Grade	Amount of asphalt					1H	2H	3H	5H	6H	8H	9H
Fine and gap graded asphalt concrete	7.0% (OAC)	Tohoku		2 × 3	2.302	0.29	0.53	0.74	-	-	1.85	-
		Kanto		2 × 3	2.314	0.68	1.10	1.57	-	-	-	-
		Hokuriku		2 × 3	2.315	0.35	0.58	0.81	1.19	-	-	-
		PWRI		2 × 2	2.318	0.51	0.75	1.03	-	1.69	-	2.23
		Hokkaido	Rotating type	2 × 3	2.305	1.07	1.73	2.27	-	-	-	-
			Shuttling type	2 × 3	2.305	1.50	2.37	3.00	-	-	-	-
	6.0% (OAC-1%)	Tohoku		2 × 3	2.258	0.56	0.76	0.98	-	-	1.91	-
		Kanto		2 × 3	2.252	1.20	2.03	2.74	-	-	-	-
		Hokuriku		2 × 3	2.261	0.66	0.98	1.33	1.99	-	-	-
		PWRI		2 × 2	2.262	0.76	1.34	1.63	-	2.78	-	3.41
		Hokkaido	Rotating type	2 × 3	2.264	1.34	2.05	2.67	-	-	-	-
			Shuttling type	2 × 3	2.264	2.12	3.31	3.88	-	-	-	-
Dense and gap graded asphalt concrete	5.8% (OAC)	Tohoku		2 × 3	2.325	0.26	0.32	0.38	-	-	0.84	-
		Kanto		2 × 3	2.317	0.58	1.01	1.30	-	-	-	-
		Hokuriku		2 × 3	2.318	0.50	0.60	0.85	0.94	-	-	-
		PWRI		2 × 2	2.320	0.43	0.68	0.87	-	1.41	-	1.80
		Hokkaido	Rotating type	2 × 3	2.319	1.00	1.53	1.84	-	-	-	-
			Shuttling type	2 × 3	2.319	1.56	2.28	2.83	-	-	-	-
	5.0% (OAC-0.8%)	Tohoku		2 × 3	2.274	0.34	0.64	0.82	-	-	1.44	-
		Kanto		2 × 3	2.268	1.37	2.09	2.44	-	-	-	-
		Hokuriku		2 × 3	2.270	0.44	0.73	1.15	1.50	-	-	-
		PWRI		2 × 2	2.267	1.15	1.74	2.09	-	3.16	-	3.82
		Hokkaido	Rotating type	2 × 3	2.263	1.60	2.17	2.79	-	-	-	-
			Shuttling type	2 × 3	2.263	2.30	3.47	4.25	-	-	-	-

Table 14 Coefficients of Conversion Based on Asphalt Pavement Outline Type

		Hokkaido shuttling side	Hokkaido shuttling side	Hokkaido revolution side	PWRI revolution side	Kanto revolution side	Hokuriku revolution side	Tohuku revolution side
Fine grade G	Area of wear (cm^2)	1.17	2.40	2.77	2.90	0.81	0.86	0.80
	Coefficient of conversion a_1	1.00	0.49	0.42	0.40	1.44	1.36	1.46
Dense grade G	Area of wear (cm^2)	1.58	2.87	3.00	2.65	0.89	0,76	0.35
	Coefficient of conversion a_2	1.00	0.55	0.53	0.60	1.78	2.08	4.51
$a_1 + a_2/2$		1.00	0.52	0.48	0.50	1.61	1.72	2.99

Table 15 Coefficients of Conversion Based on Shuttling Type of Hokkaido (Test time: 3 hours)

		Hokkaido		Kanto	PWRI	Hokuriku	Tohoku
		Shuttling type	Revolution type				
A-1	A (cm^2)	3.00	2.27	1.57	1.03	0.81	0.74
	a	1.00	1.32	1.91	2.91	3.70	4.05
A-2	A	3.88	2.67	2.74	1.63	1.33	0.98
	a	1.00	1.45	1.42	2.38	2.92	3.96
B-1	A	2.83	1.84	1.30	0.87	0.85	0.38
	a	1.00	1.54	2.18	3.25	3.33	7.45
B-2	A	4.25	2.79	2.44	2.09	1.15	0.82
	a	1.00	1.52	1.74	2.03	3.70	5.18
Mean $\bar{a}$		1.00	1.46	1.81	2.64	3.41	5.16

*A: Area of wear a: Coefficient of conversion

Table 16 Test Time Corresponding to One Hour and a Half of Asphalt Pavement Outline

Organizations / Test samples	Kanto	Hokuriku	Kanto	PWRI	Hokkaido (revolution type)
A-1	(17.5)	(9.5)	(4.0)	7.2	2.3
A-2	(17.5)	(8.0)	2.9	5.7	3.0
B-1	-	-	(5.5)	(9.5)	3.3
B-2	-	-	(4.0)	5.2	3.1
Mean $\sqrt{V}$	17.5 0	8.8 0.8	4.1 0.9	6.9 1.7	2.9 0.4
Utility value	18	9	4	6	3

( ): Presumed value by external insertion

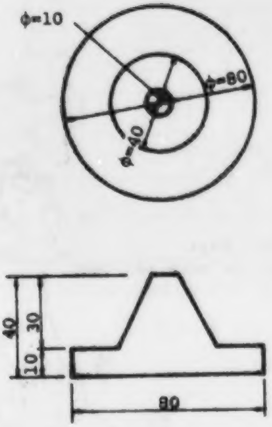

φ=10
φ=80
φ=40
40
30
10
80

Fig. 1 Shape of Wear Pin (mm)

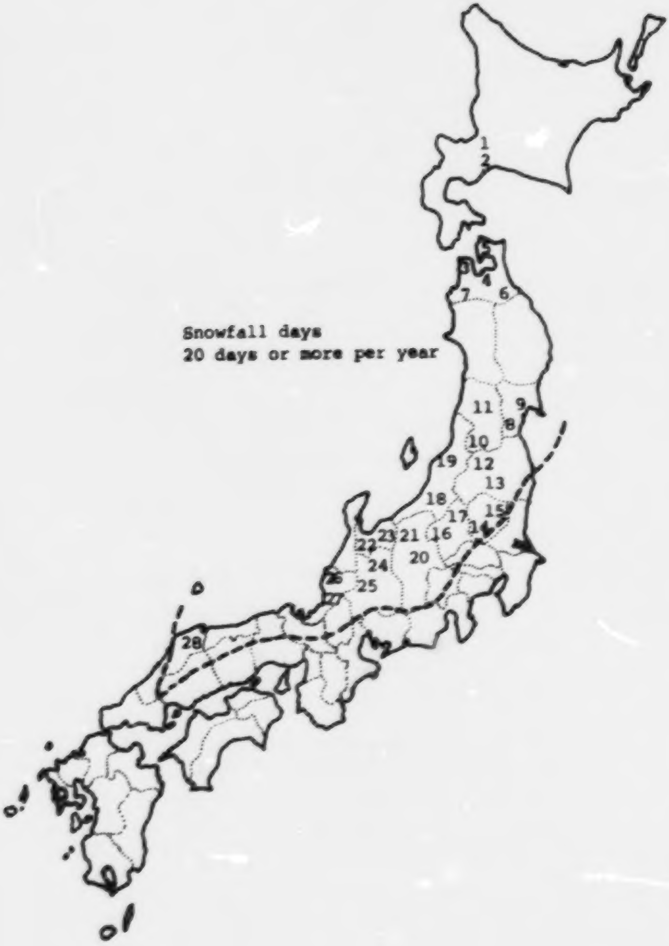

Snowfall days
20 days or more per year
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
21
22
23
24
25
26
27
28

Fig. 2 Number of Research Points

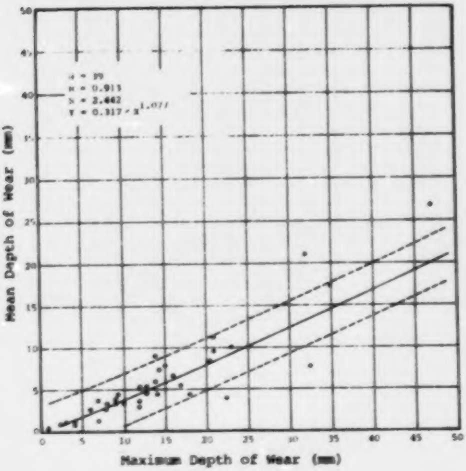


Fig. 3

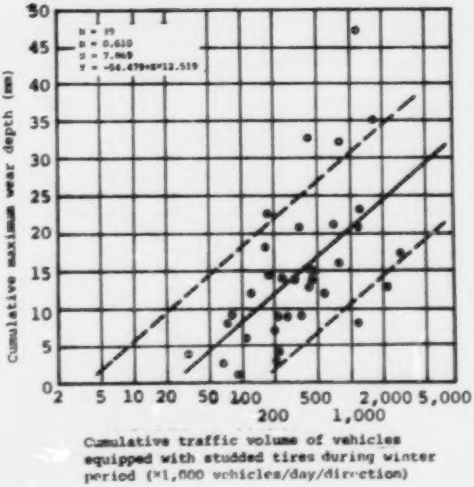


Fig. 4 Relationship between Cumulative Traffic Volumes of Vehicles Equipped with Studded Tires During Winter Period and Depths of Wear

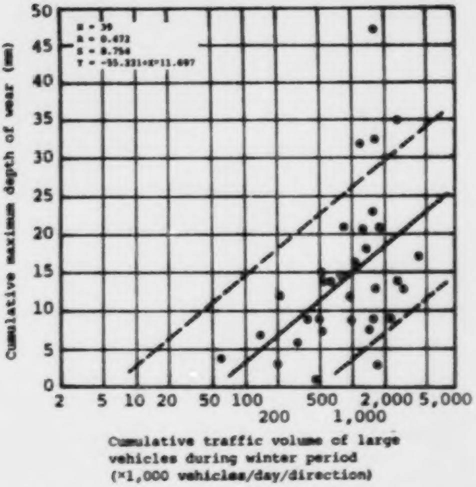


Fig. 5 Relationship between Cumulative Traffic Volumes During Winter Period and Depths of Wear

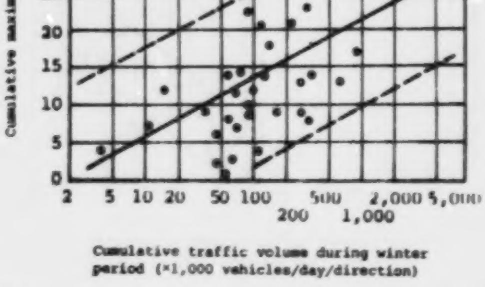


Fig. 6 Relationship between Cumulative Traffic Volume of Large Vehicles During Winter Period and Depths of Wear

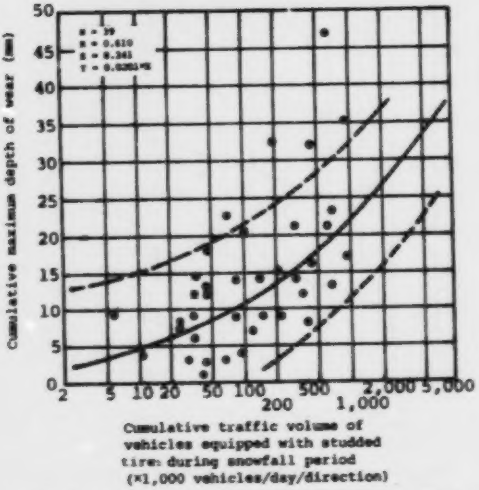


Fig. 7 Relationship between Cumulative Traffic Volumes of Vehicles Equipped with Studded Tires During Snowfall Period and Wear Depths

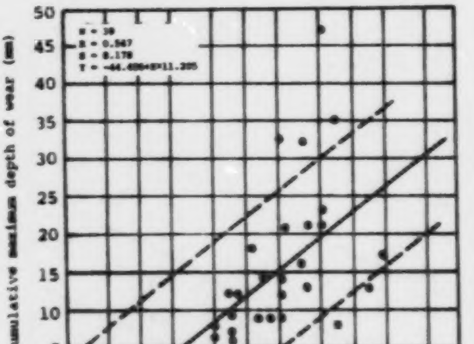


Fig. 8 Relationship between Cumulative Traffic Volumes of Vehicles Equipped with Studded Tires During Snow Removal Period and Wear Depths

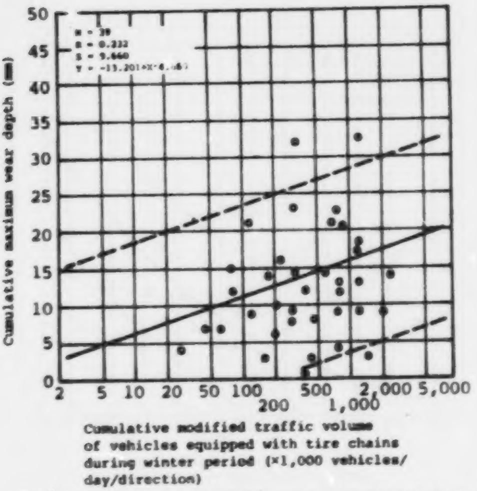


Fig. 9 Relationship between Cumulative Modified Traffic Volumes of Vehicles Equipped with Tire Chains During Winter Period and Wear Depth

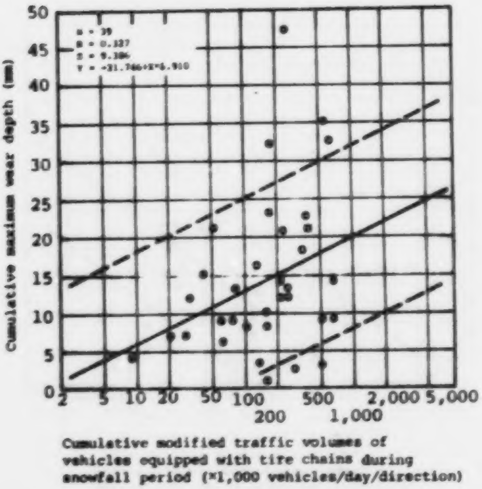


Fig. 10 Relationship between Cumulative Modified Traffic Volumes of Vehicles Equipped with Tire Chains During Snowfall Period and Wear Depths

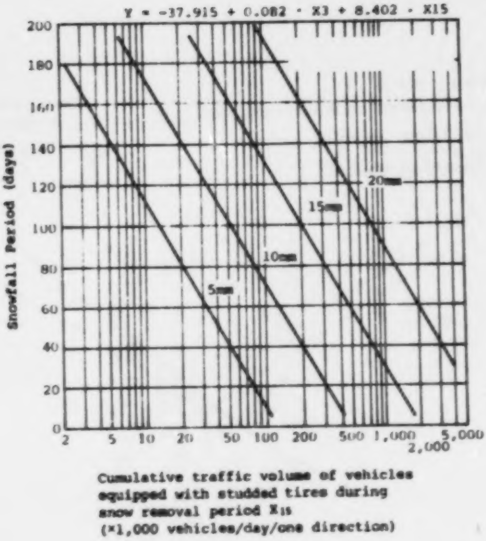

Y = -37.915 + 0.082 · X3 + 8.402 · X15
Snowfall Period (days)
200
180
160
140
120
100
80
60
40
20
0
2
5
10
20
50
100
200
500
1,000
2,000
5,000
5mm
10mm
15mm
20mm
Cumulative traffic volume of vehicles equipped with studded tires during snow removal period X15
(×1,000 vehicles/day/one direction)

Fig. 11

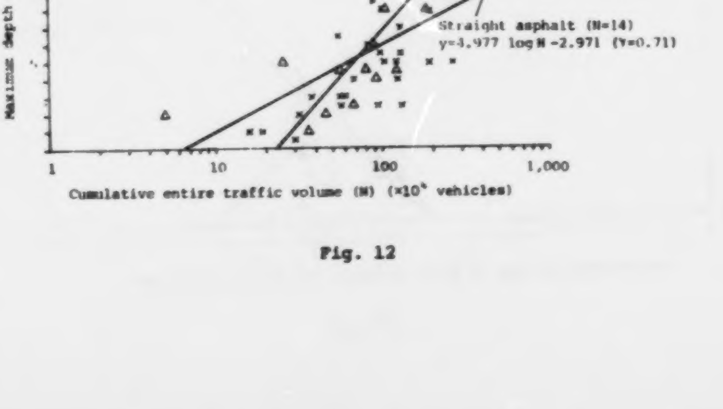

Hokkaido region
(Fine graded asphalt concrete 13F)
Δ Straight asphalt
x Modified asphalt
Rubberized asphalt (N=37)
y=10.528 log N - 13.343
Straight asphalt (N=14)
y=4.977 log N -2.971 (Y=0.71)
Maximum depth of wear (Y) (mm)
20
15
10
5
1
10
100
1,000
Cumulative entire traffic volume (N) (×10⁴ vehicles)

Fig. 12

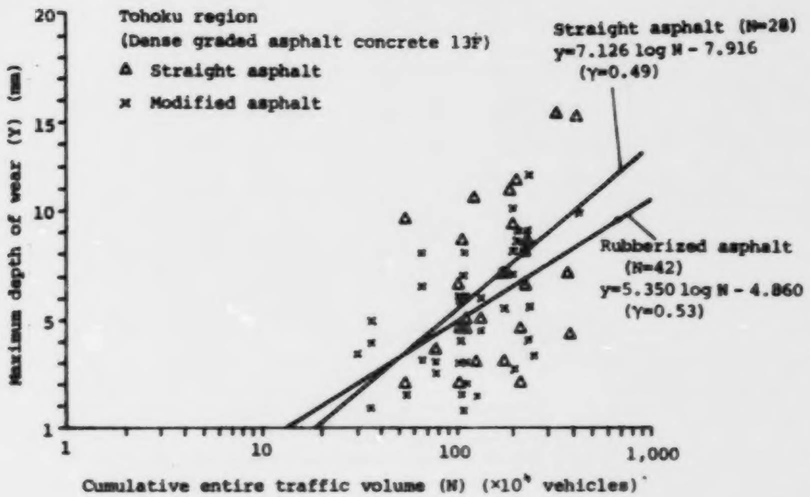

Tohoku region
(Dense graded asphalt concrete 13F)
Δ Straight asphalt
× Modified asphalt
Straight asphalt (N=28)
y=7.126 log N - 7.916
(γ=0.49)
Rubberized asphalt
(N=42)
y=5.350 log N - 4.860
(γ=0.53)
Maximum depth of wear (Y) (mm)
20
15
10
5
1
1
10
100
1,000
Cumulative entire traffic volume (N) (×10⁴ vehicles)

Fig. 13

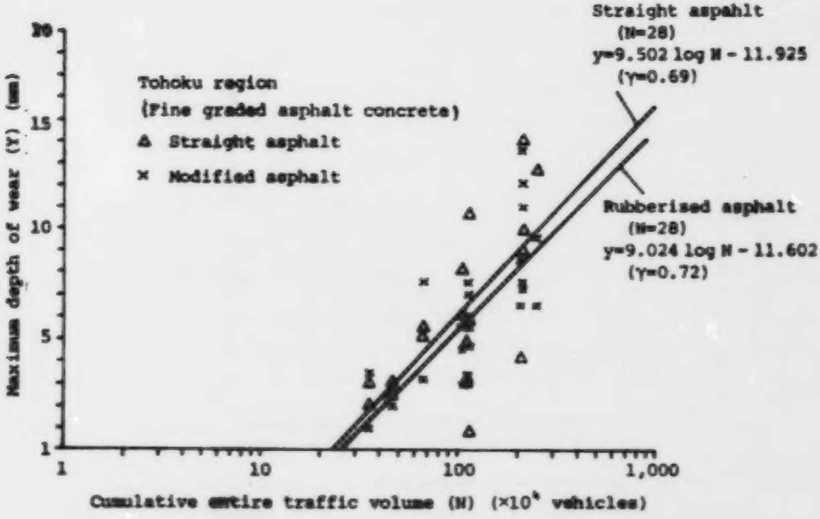

Straight aspahlt
(N=28)
y=9.502 log N - 11.925
(γ=0.69)
Tohoku region
(Fine graded asphalt concrete)
Δ Straight asphalt
× Modified asphalt
Rubberised asphalt
(N=28)
y=9.024 log N - 11.602
(γ=0.72)
Maximum depth of wear (Y) (mm)
20
15
10
5
1
1
10
100
1,000
Cumulative entire traffic volume (N) (×10⁴ vehicles)

Fig. 14

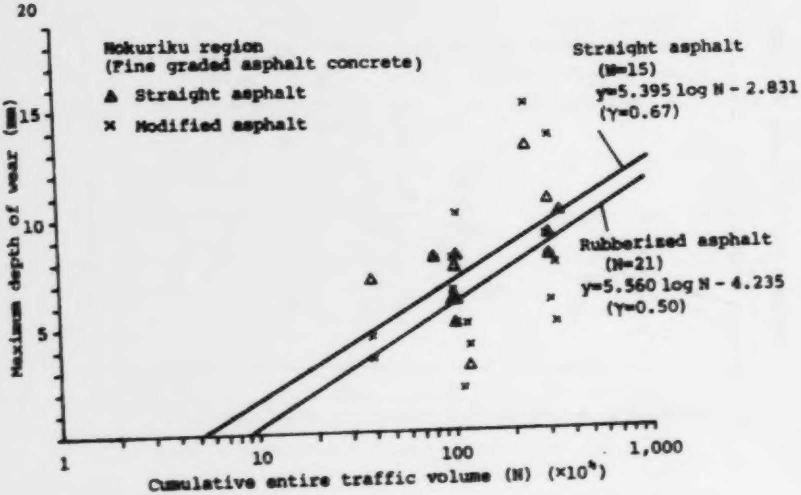

20
15
10
5
Maximum depth of wear (mm)
Hokuriku region
(Fine graded asphalt concrete)
▲ Straight asphalt
× Modified asphalt
Straight asphalt
(N=15)
y=5.395 log N - 2.831
(γ=0.67)
Rubberized asphalt
(N=21)
y=5.560 log N - 4.235
(γ=0.50)
1
10
100
1,000
Cumulative entire traffic volume (N) (×10⁴)

Fig. 15

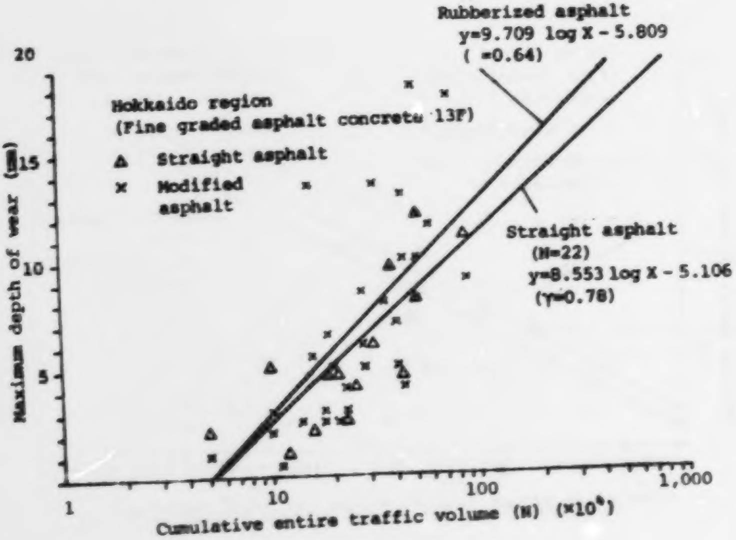

Rubberized asphalt
y=9.709 log X - 5.809
(=0.64)
20
15
10
5
Maximum depth of wear (mm)
Hokkaido region
(Fine graded asphalt concrete 13F)
Δ Straight asphalt
× Modified asphalt
Straight asphalt
(N=22)
y=8.553 log X - 5.106
(γ=0.78)
1
10
100
1,000
Cumulative entire traffic volume (N) (×10⁴)

Fig. 16

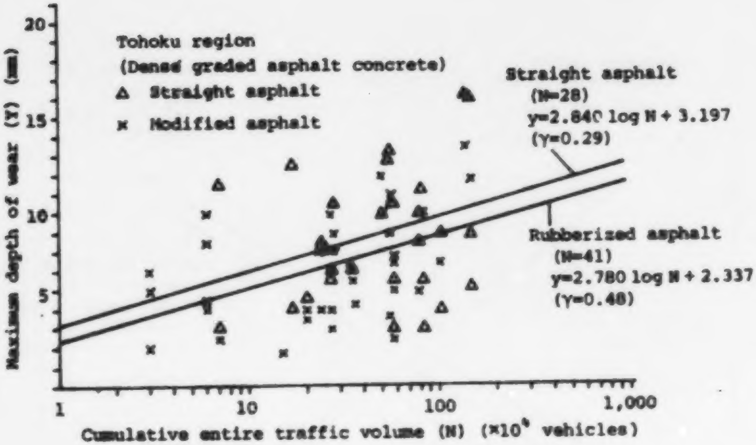

Tohoku region
(Dense graded asphalt concrete)
Δ Straight asphalt
× Modified asphalt
Straight asphalt
(N=28)
y=2.840 log N + 3.197
(γ=0.29)
Rubberized asphalt
(N=41)
y=2.780 log N + 2.337
(γ=0.48)
Maximum depth of wear (Y) (mm)
20
15
10
5
1
10
100
1,000
Cumulative entire traffic volume (N) (×10⁴ vehicles)

Fig. 17

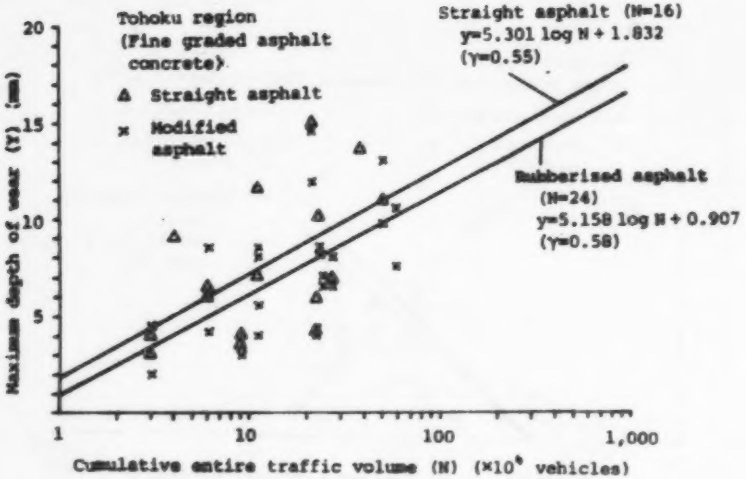

Tohoku region
(Fine graded asphalt concrete)
Δ Straight asphalt
× Modified asphalt
Straight asphalt (N=16)
y=5.301 log N + 1.832
(γ=0.55)
Rubberized asphalt
(N=24)
y=5.158 log N + 0.907
(γ=0.58)
Maximum depth of wear (Y) (mm)
20
15
10
5
1
10
100
1,000
Cumulative entire traffic volume (N) (×10⁴ vehicles)

Fig. 18

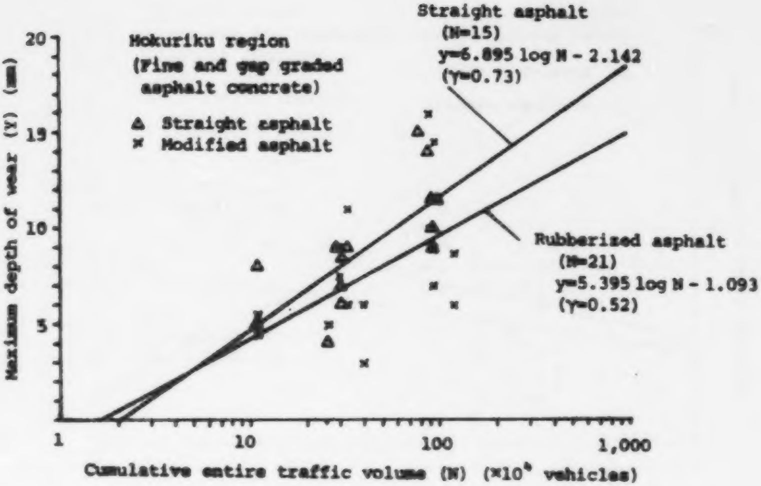

Hokuriku region
(Fine and gap graded asphalt concrete)
△ Straight asphalt
✕ Modified asphalt
Straight asphalt
(N=15)
y=6.895 log N - 2.142
(γ=0.73)
Rubberized asphalt
(N=21)
y=5.395 log N - 1.093
(γ=0.52)
Maximum depth of wear (Y) (mm)
20
15
10
5
1
10
100
1,000
Cumulative entire traffic volume (N) (×10⁴ vehicles)

Fig. 19

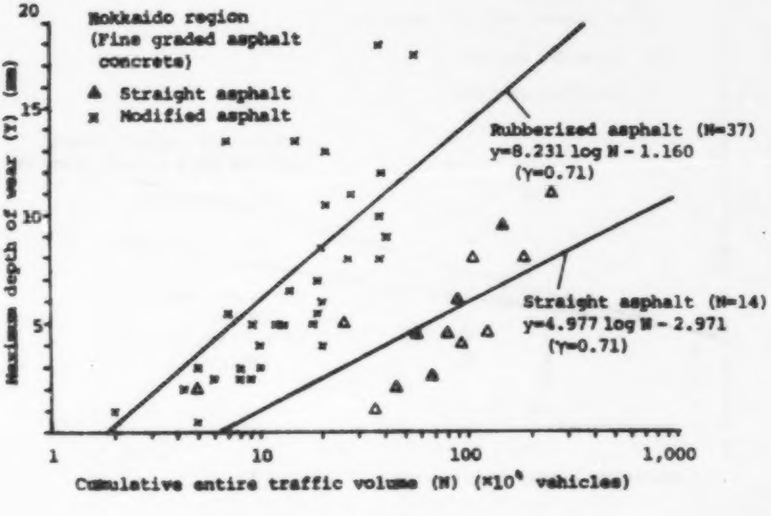

Hokkaido region
(Fine graded asphalt concrete)
▲ Straight asphalt
✕ Modified asphalt
Rubberized asphalt (N=37)
y=8.231 log N - 1.160
(γ=0.71)
Straight asphalt (N=14)
y=4.977 log N - 2.971
(γ=0.71)
Maximum depth of wear (Y) (mm)
20
15
10
5
1
10
100
1,000
Cumulative entire traffic volume (N) (×10⁴ vehicles)

Fig. 20

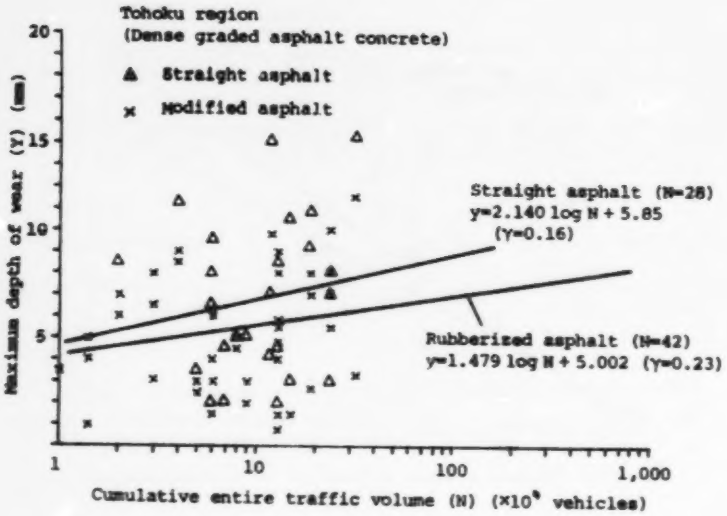

Tohoku region
(Dense graded asphalt concrete)
△ Straight asphalt
× Modified asphalt
Straight asphalt (N=28)
y=2.140 log N + 5.85
(γ=0.16)
Rubberized asphalt (N=42)
y=1.479 log N + 5.002 (γ=0.23)
Maximum depth of wear (Y) (mm)
20
15
10
5
1
10
100
1,000
Cumulative entire traffic volume (N) (×10⁹ vehicles)

Fig. 21

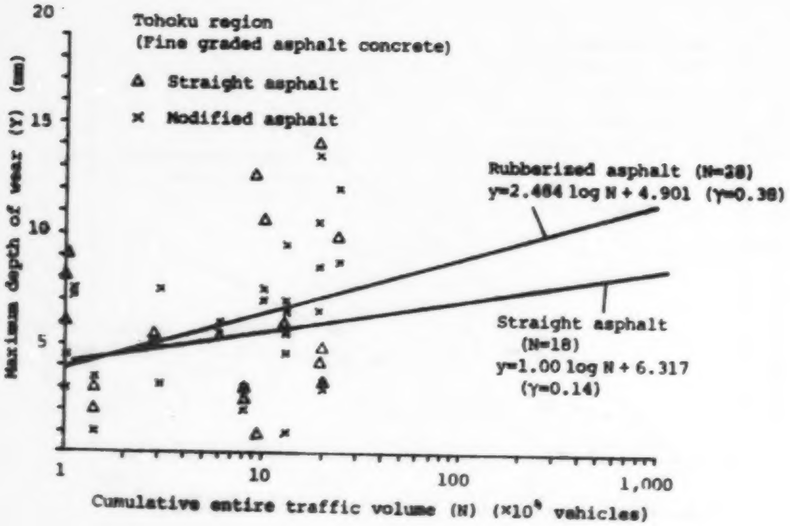

Tohoku region
(Fine graded asphalt concrete)
△ Straight asphalt
× Modified asphalt
Rubberized asphalt (N=28)
y=2.484 log N + 4.901 (γ=0.38)
Straight asphalt
(N=18)
y=1.00 log N + 6.317
(γ=0.14)
Maximum depth of wear (Y) (mm)
20
15
10
5
1
10
100
1,000
Cumulative entire traffic volume (N) (×10⁹ vehicles)

Fig. 22

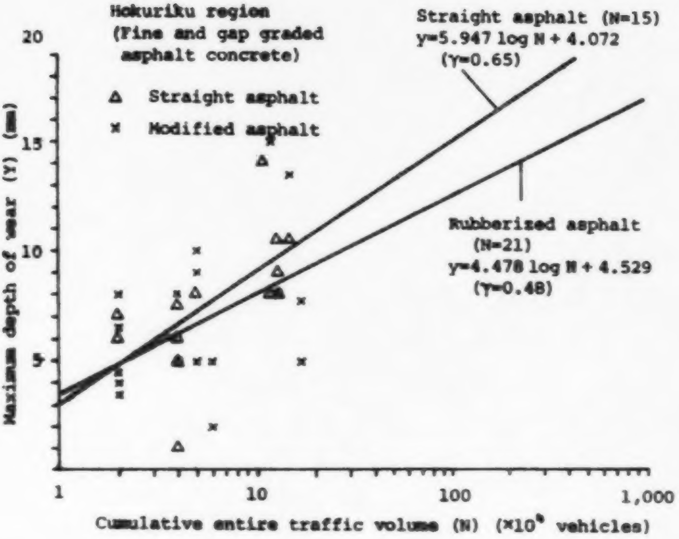

Hokuriku region
(Fine and gap graded asphalt concrete)
Δ Straight asphalt
× Modified asphalt
Straight asphalt (N=15)
$y=5.947 \log N + 4.072$
$(\gamma=0.65)$
Rubberized asphalt
(N=21)
$y=4.478 \log N + 4.529$
$(\gamma=0.48)$
20
15
10
5
Maximum depth of wear (Y) (mm)
1
10
100
1,000
Cumulative entire traffic volume (N) ($\times 10^6$ vehicles)

Fig. 23

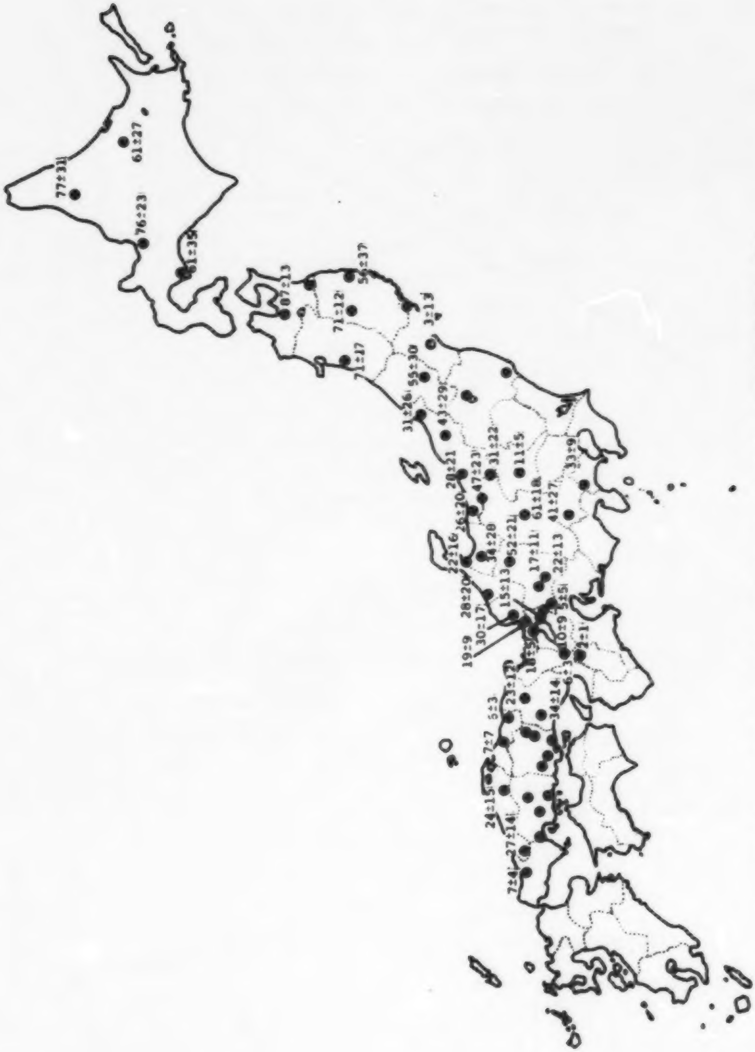

77±31
61±27
76±23
81±39
87±13
71±12
54±37
71±17
3±13
55±30
31±26
43±29
28±21
47±23
31±22
11±5
26±20
61±18
33±9
34±28
52±24
41±27
22±16
17±11
22±13
28±20
15±13
30±17
19±9
10±9
3±5
2±1
23±17
34±14
5±3
7±7
24±15
27±14
7±4

Fig. 24 Rate of Small Vehicles Equipped with Studded Tires (%)
($\bar{x}$±16 research from November 1981 to March 1982)

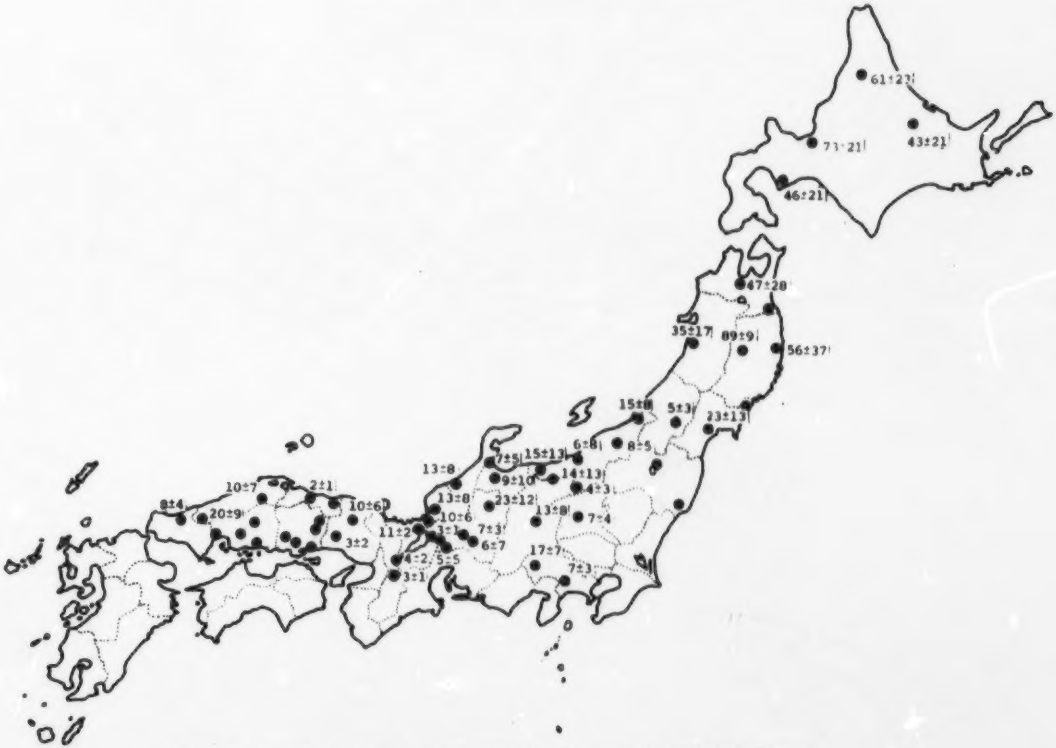

61±23
73±21
43±21
46±21
47±28
35±17
89±9
56±37
15±8
5±3
23±13
6±8
8±5
7±5
15±13
13±8
9±10
14±13
4±3
13±8
23±12
13±8
7±4
10±6
3±1
7±3
6±7
11±2
4±2
5±5
3±1
17±7
7±3
10±7
2±1
8±4
20±9
10±6
3±2

Fig. 25 Rate of Large Vehicles Equipped with Studded Tires During Winter Period (%)
($\bar{x}\pm1\sigma$ research from November 1981 to March 1982)

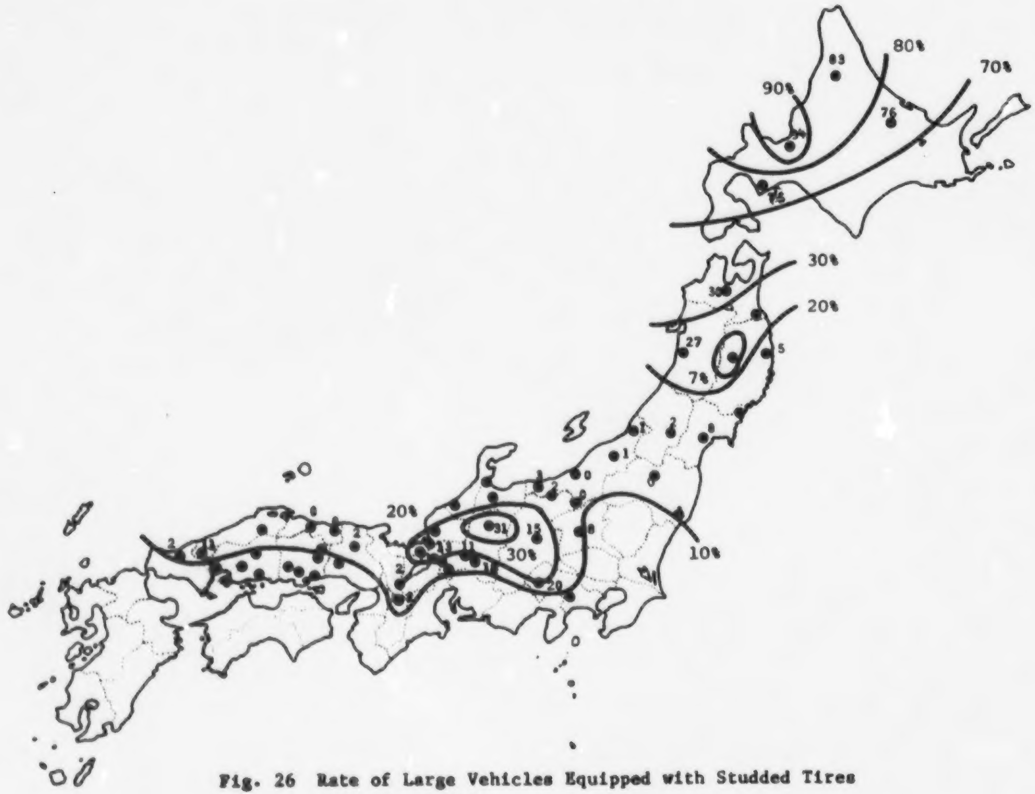

90%
80%
70%
83
76
75
30%
20%
30
27
7%
5
1
0
2
20%
31
15
30%
11
20
2
10%

Fig. 26 Rate of Large Vehicles Equipped with Studded Tires
(November 1981)
($\bar{x}$±16 in survey during November 1981 to March 1982)

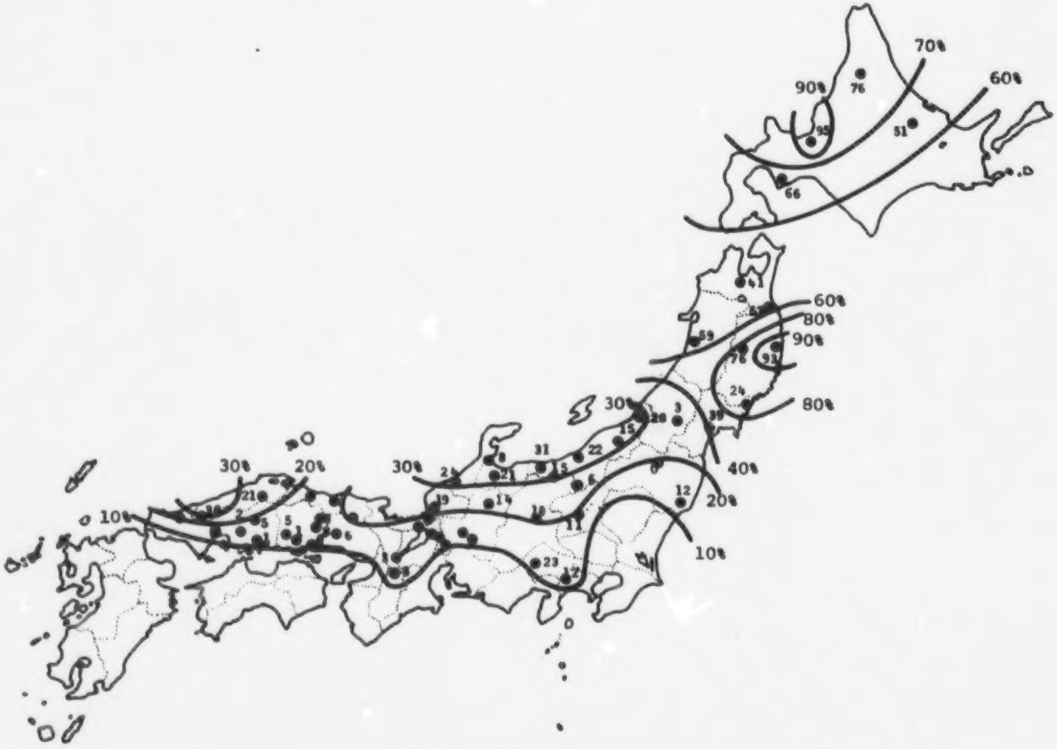

70%
60%
90%
76
51
95
66
41
60%
80%
90%
93
78
24
80%
59
30%
28
3
38
15
22
31
40%
20%
12
10%
30%
24
8
21
5
6
19
14
18
11
23
12
30%
20%
21
10%

Fig. 27 Rate of Large Vehicles Equipped with Studded Tires (January 1982)

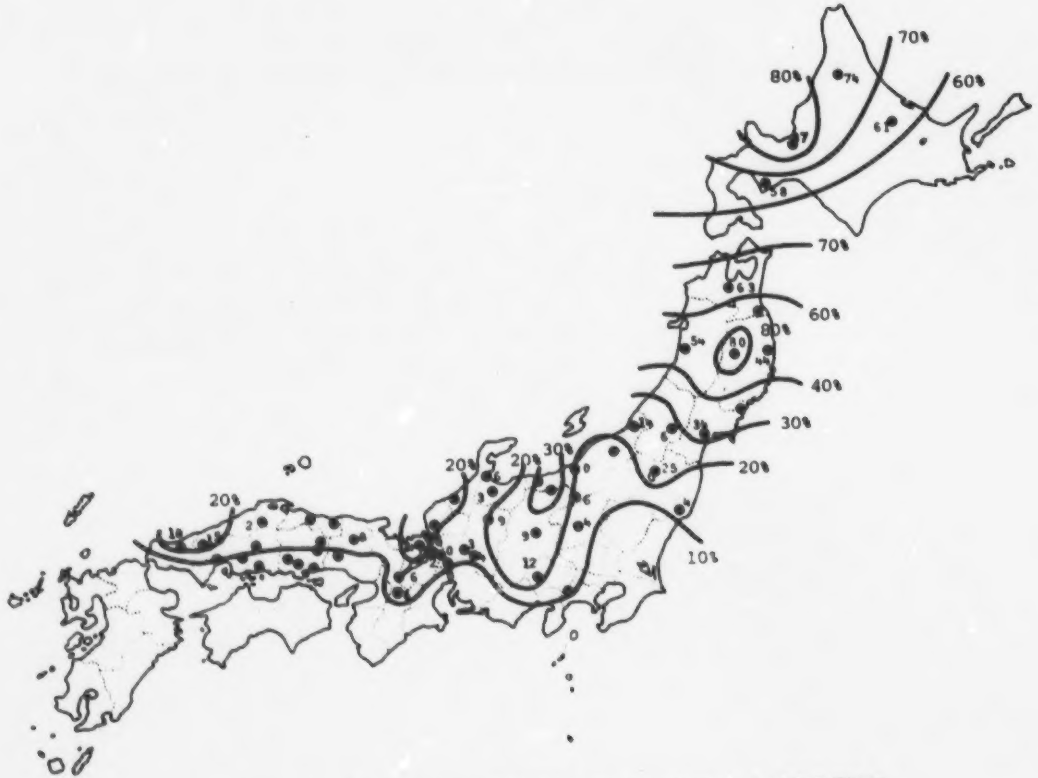

80%
70%
60%
74
67
61
58
70%
63
60%
80%
80
54
40%
30%
30%
20%
25
10%
20%
20%
30%
20%
12
9
6
20%

Fig. 28 Rate of Large Vehicles Equipped with Studded Tires
(March 1982)

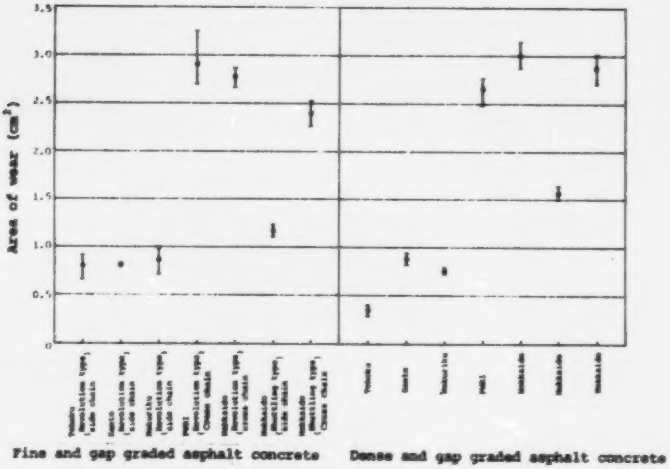
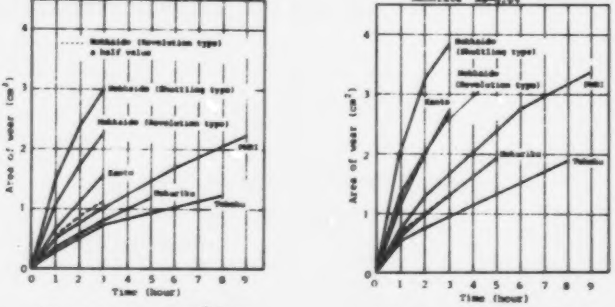


Fig. 29 Test Result 1979

A-1 Fine and gap graded asphalt concrete As=7.0%

A-2 Fine and gap graded asphalt concrete As=6.0%



B-1 Dense and gap graded asphalt concrete As=5.8%

B-2 Dense and gap graded asphalt concrete As=5.0%

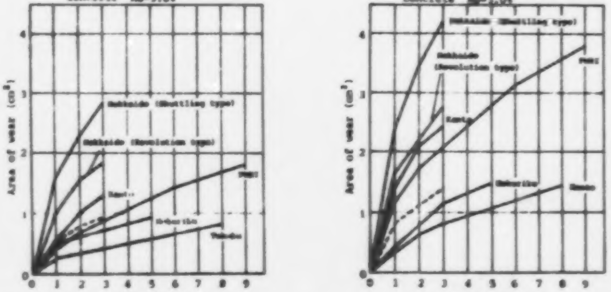


Fig. 30 Test Result 1980

Paving
in Cold
Areas

Vancouver B.C., 1982

Session 5

Characterization of Flexible Pavement Components

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CHARACTERIZATION OF FLEXIBLE PAVEMENT COMPONENTS

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Trow Ltd. Consulting Engineers

ABSTRACT

The full characterization of flexible pavement components must involve consideration of both structural capacity and drainage requirements. The resilient modulus (M_R) and resilient Poisson's ratio (ν_R) characterization of asphaltic concrete surface and binder course mixes (laboratory prepared specimens and cores of various ages), granular base and sub-base aggregates (various types and use categories) and typical subgrade materials is described in terms of equipment development, testing procedures and general results, with a comparison to typical procedures and findings from the literature. Simple diametral resilient modulus equipment was adopted for bound materials, but the importance of confining influences required dynamic triaxial testing for unbound materials. For the base and subgrade materials, the important interacting permeability (k) aspects were also considered, which involved permeability testing of CBR size specimens prepared to conditions representative of field situations. Stress levels and environmental conditions typical to Ontario were adopted.

The research results are summarized, with particular emphasis on the innovative aspects of equipment development and test methods. Key findings such as aging influences on asphaltic concrete M_R which tend to be greater than differences between mixes, and M_R and k ranges for granular materials, are discussed. It is concluded that the use of dense graded granular bases and sub-bases may optimize structural capacity, but this is often at the expense of the free-draining behaviour essential to long-term pavement performance.

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CHARACTERIZATION OF FLEXIBLE PAVEMENT COMPONENTS

JOHN EMERY

TROW LTD.

INTRODUCTION

With the trend towards rational design of flexible pavements through numerical analyses of layered systems for the strain, stress and fatigue regime at key points within the pavement structure, rather than empirical designs, it is important that representative data are available on materials characteristics, anticipated loadings and environmental conditions. Typical rational computerized approaches for design and pavement performance prediction are based on elastic theory, even though pavement materials are non-linear time-dependent, highly temperature-dependent in the case of asphaltic concretes and clearly influenced by the field environment, particularly the action of water. However, for the rates of loading, stress levels and recovery times available, the concept of working with recoverable deformations during dynamic characterization [resilient modulus, M_R (repeated stress/resilient strain) and resilient Poisson's ratio, ν_R (resilient lateral strain/resilient vertical strain)] is generally accepted. Adopting coupled non-linear visocoelastic-thermal - moisture condition analyses, say using the finite element method, would require flexible pavement components characterization capabilities and data far in advance of those currently available, even for use with elastic theory on a regular basis.

The study reported herein, completed at McMaster University with Ontario Ministry of Transportation and Communications (MTC) sponsorship, was developed to provide resilient moduli (M_R and ν_R) for asphaltic concretes, base and subbase aggregates and subgrade soils typical to Ontario. Recognizing the key role of water to flexible pavement performance, i.e. the need for adequate pavement structure drainage, permeability coefficients (k) were determined for the granular aggregates. Provision of characterization data was the prime objective, but it was also recognized that the diametral and triaxial testing equipment and procedures developed must be suitable for regular use during pavement design. It should be noted that the testing and data described throughout are for sub-failure levels with minimal permanent deformations and no consideration of fatigue. These are obvious important extensions to the study.

FACTORS INFLUENCING RESILIENT MODULI AND TYPICAL VALUES

As a first stage of the study, a literature survey was completed in terms of factors influencing resilient moduli, response, characterization procedures and representative moduli (1-4 and their references, for instance):

Bound Materials

-asphaltic concretes and stabilized bases

·response approaches linear elastic for loadings involved

·characterization typically with dynamic diametral or triaxial equipment.

Asphaltic Concretes

-factors influencing M_R and ν_R

·primary - temperature, the log M_R - temperature relationship is essentially linear for typical mixes

·secondary-mix variables (asphalt cement stiffness, asphalt cement content, density, nature and grading of aggregates, aging)

-loading variables (magnitude, rate, frequency and duration of vertical or deviatoric stress applications, rest periods, confining stress level to some extent)

-testing variables (laboratory prepared specimen or field specimen).

-representative M_R and ν_R

·M_R- 1.4×10^6 to 4.1×10^6 kPa

·ν_R- 0.30 to 0.50

Asphalt Stabilized Bases

-factors influencing M_R and ν_R

·in addition to those for asphaltic concrete - curing time and conditions for emulsified and cut-back asphalts, care in handling lower strength specimens.

-representative M_R

·emulsified asphalt stabilization - 1.9×10^5 to 2.9×10^5 kPa (21°C, confining stress of 55 to 207 kPa).

Cement and Lime/Fly Ash Stabilized Bases

-factors influencing M_R and ν_R

·cement or lime/fly ash content, nature and grading of aggregates, age.

-representative M_R and ν_R

·M_R- 3.4×10^6 to 1.0×10^7 kPa

·ν_R - 0.10 to 0.25

Unbound Materials

-granular materials and cohesive soils (mainly subgrades)

·response non-linear elastic for loadings involved

·characterization typically with dynamic triaxial equipment.

Granular Aggregates

-factors influencing M_R and ν_R

·primary - confining stress level, typically through non-linear M_R relationship of the form $M_R = A\theta^B$ where θ is the first stress invariant in triaxial tests, A and B are constants based on the tests.

·secondary-density, gradation, particle characteristics such as angularity, degree of saturation, preparation procedures (no inherent strength).

-representative M_R and ν_R

·granular base M_R - 1.0×10^5 to 6.9×10^5 kPa

ν_R - 0.20 to 0.50 (?)

·granular sub-base M_R - 6.9×10^4 to 2.8×10^5 kPa

ν_R - 0.20 to 0.50 (?)

Cohesive Soils (Subgrades)

-factors influencing M_R and ν_R

·soil characteristics (plasticity, -75 µm content, etc.), moisture content, density, sampling of soil and preparation procedures. Attempts have been made to relate M_R to CBR, but the stress and deformation conditions are not comparable.

-representative M_R and ν_R, clay to fine-grained sandy silt

·M_R - 2.1×10^4 to 2.1×10^5 kPa

·ν_R - 0.30 to 0.50

From this review, several key aspects became apparent that were considered during the characterization of each flexible pavement component: M_R values adopted in Ontario Pavement Analysis of Costs (OPAC) appear to be representative with only refinement required (5,6); influence of in-service aging that results in stiffer asphaltic concretes must be considered; open graded, recycled and stabilized asphaltic concretes require characterization; specimen preparation and testing of granular aggregates and subgrade soils must accommodate low inherent strengths and key influence of confinement on response; and there are important interacting aspects of materials behaviour, particularly the important role of water and frost in the pavement structure.

CHARACTERIZATION OF ASPHALTIC CONCRETES

Equipment

The characterization of asphaltic concretes was completed using both the dynamic diametral tension method developed by Schmidt (7) with the addition of temperature and confinement control in a triaxial cell, and the dynamic triaxial method similar to that adopted in soil mechanics, as shown schematically in Figure 1 (8). In the diametral test on Marshall briquette size specimens, sub-yield, dynamic line loads (distributed over a 19 mm wide strip) are applied along the vertical axis with the lateral displacement response measured across the horizontal axis. The resilient modulus for a 102 mm diameter cylindrical specimen is then given by:

$$M_R = \frac{P}{t\Delta} (0.9942\ \nu_R + 0.2642) \qquad (1)$$

where P is the applied load; t the specimen thickness; Δ the total resilient (instantaneously recovered) deformation across the diameter perpendicular to loading; and ν_R the resilient Poisson's ratio. The simple diametral equipment was modified so that it could be mounted in a 178 mm diameter triaxial cell with the main features shown in Figure 1 for a triaxial specimen; axial load monitoring with an internal load cell; displacement monitoring (vertical and lateral) with cantilever strain gauges shown in more detail for a triaxial specimen in Figure 2: heating oil supply with temperature control system; vertical load and confining pressure application and timing system; and data acquisition system. This triaxial equipment was extended throughout the study to accommodate granular aggregates and subgrade soils, reaching the stage of a micro-processor to control loading and monitoring.

To determine the resilient Poisson's ratio, and compare dynamic diametral results with triaxial results, the measuring device shown in Figure 2 was adopted. Vertical and horizontal stress levels for testing were based on BISTRO analyses of typical flexible pavement sections for 80 kN axle loadings with tire pressures of 550 kPa (σ_V = 186 kPa and σ_h = 97 kPa at the bottom of a 102 mm thick asphaltic concrete surface). A dynamic loading duration of 0.1 s at a frequency of 20 cpm was chosen as this represents truck creep speeds. The 2.9 s rest period between approximately haversine load pulses permits viscoelastic recovery. A temperature range of about 0 to 50°C was used to represent typical flexible pavement temperatures during the critical spring thaw and hot summer periods. More complete details on the equipment and early testing are given in Reference 8.

Test Results

Diametral Resilient Moduli

To obtain a realistic range of asphaltic concrete specimens for characterization, both Marshall type specimens (briquettes 102 mm in diameter and 64 mm thick) and triaxial type specimens (102 mm in diameter and 204 mm in height) were supplied by the MTC covering "excellent", "average" and "marginal" mix designs based on their experience. All Marshall type specimens were tested in the diametral resilient modulus equipment with the typical M_R results for "excellent" surface course (HL-3) mixes given in Figure 3, for assumed ν_R values from previous testing: 0.25 for 0 to $10^{\circ}C$; 0.35 for 18 to $23^{\circ}C$; and 0.45 for 38 to $43^{\circ}C$. The test temperature was achieved by holding the briquette at the desired temperature and then rapidly testing it in the diametral equipment. This diametral resilient modulus testing for a range of mix qualities indicated that the optimum M_R occurs at lower asphalt cement contents than optimum Marshall designs for durability and stability would indicate, with the predominant influence of temperature showing up clearly. It was found that the M_R values determined were somewhat less than those adopted in OPAC, with little difference in M_R related to mix quality - Figure 4 for basecourse mixes for instance.

Three of the surface course (HL-3) Marshall specimens were also tested with various intensities of both dynamic diametral load and pulsed confining pressure in the triaxial cell with close temperature control. These tests confirmed trends observed in preliminary testing (8): there is little difference between conventional and triaxial diametral resilient moduli, i.e. the simple Schmidt apparatus and approach (7) are adequate for regular design characterization of asphaltic concretes; for variations in dynamic diametral load representative of traffic loadings, the M_R varies less than about 4 percent for design temperatures at sub-permanent deformation levels; and increasing the pulsing confining pressure causes a moderate M_R increase (about 30 percent for a 48 kPa confining pressure increase at $20^{\circ}C$), but this is only a secondary effect compared to the influence of temperature on M_R.

Triaxial Resilient Moduli

The triaxial type specimens were tested in the triaxial resilient modulus equipment with the measuring device shown in Figure 2. Although specifically developed to determine ν_R, this equipment also gives M_R directly and is compatible with ASTM procedures. The ν_R-temperature relationships for several mixes are given in Figure 5, demonstrating a greater range than anticipated from the literature. Although the primary rational design parameters for flexible pavements (maximum compressive subgrade strain and maximum tensile asphaltic concrete strain) are relatively insensitive to variations in ν_R, the calculation of M_R as determined by the diametral method is appreciably affected by ν_R making the data in Figure 5 most useful. The higher ν_R values at elevated temperatures for lower quality mixes (ν_R approaches 0.5 for an incompressible fluid) are clearly an indicator of deformation susceptibility. It should be noted that steel slag asphaltic concrete mixes typically demonstrate high, in-service stability.

The log M_R - temperature plots for all triaxial type specimens are given in Figure 6 indicating an approximately straight line relationship again. As found for Marshall briquettes, the M_R for "excellent", "average" and "marginal" mixes for each course at optimum Marshall design asphalt cement content were fairly close despite their appreciable differences in Marshall properties. It was also found that there was close agreement between M_R determined for similar briquette and triaxial specimens as summarized in Table 1, except at higher temperatures where the equation for diametral M_R based on elastic theory is not applicable due to viscous effects.

General Findings For Laboratory Prepared Specimens

From the M_R and ν_R testing of laboratory prepared briquettes and triaxial specimens, it is possible to develop general characterization input for design use. Although there is appreciable variability in ν_R with mixes, the median values for the laboratory mixes can be adopted without introducing serious error since ν_R is of secondary importance to design: 0.25 near freezing (0 to 2°C); 0.38 at room temperature (20 to 23°C); and 0.43 at elevated temperature (approximately 40°C). For any temperature within the range 18 to 32°C, M_R may be estimated from:

$$M_{R2} = (M_{R1})\ (0.894)^{(T_2-T_1)} \quad (r^2 = 0.988) \qquad (2)$$

where M_{R2} is the resilient modulus (MPa) for required temperature T_2 (°C); and M_{R1} is the resilient modulus (MPa) measured at temperature T_1 (°C). It is convenient to use Equation 2 for approximation of M_R as M_{R1} can be readily measured in the simple Schmidt apparatus (7).

Resilient Moduli of Cored Specimens

The previous M_R and ν_R characterization for laboratory prepared specimens did not involve any consideration of in-service aging that would increase the mix stiffness. A number of asphaltic concrete cores were taken from pavements in the Hamilton area of known type and age. These cores were then tested in the diametral and triaxial resilient modulus equipment. For instance, the ν_R - temperature relationship for one year old basecourse mix (HM-5, similar to MTC HL-6) is given in Figure 5, indicating fairly low values. The M_R - temperature relationships for a number of basecourse mixes (HM-5) of various ages are given in Figure 7 showing a trend to stiffening (age-hardening) with in-service life, although there is no apparent quantitative trend as the mixes were probably not the same to start with. The rate at which an asphaltic concrete stiffens in-service will be controlled by many mix and environmental related factors, and there appears to be some curvature in the log M_R - temperature relationships for cores from actual pavements. For these reasons, it is difficult to predict the M_R and ν_R of in-service asphaltic concretes without laboratory evaluations and/or knowledge of their initial condition and M_R. However, it is clear that this in-service stiffening moves the M_R upwards into the range adopted for OPAC.

While beyond the scope of this paper, it was also shown that the M_R relationships developed could be related to the Shell nomographic (design) procedure for determining stiffness. In addition the M_R, and factors influencing it, was determined for open graded asphaltic concrete, asphaltic concrete that had rutted, recycled asphaltic concrete, asphaltic concrete with carbon added, filler additions and artificially aged asphaltic concretes.

CHARACTERIZATION OF GRANULAR AGGREGATES

Equipment

As a next step in characterizing flexible pavement components, modifications were made to the dynamic triaxial resilient modulus equipment so that granular aggregates typically used in base and sub-base construction could be tested. Since unconfined granular aggregates do not have any inherent strength, a freezing procedure was adopted during preparation, and non-contact eddy current probes were used to measure vertical and lateral deformations (three spaced equidistant around the circumference). The testing system is shown schematically in Figure 8 with a photograph for a granular specimen in the cell in Figure 9. Careful specimen preparation in the laboratory was found to be the key to the overall characterization of granular aggregates;

1. Determine maximum dry density and optimum moisture content for specimen preparation using modified Proctor procedures;

2. Compact specimen in a 102 mm diameter by 203 mm high split mold with same compactive effort and moisture content as modified Proctor;

3. Place in freezer at -35°C for about two hours (no specimen disturbance observed);

4. Extract frozen specimen from split mold and place on triaxial cell base;

5. Using conventional soil testing methods, place a rubber membrane around the specimen and seal with 'O' rings;

6. Place small stainless steel targets on specimen for non-contact eddy current probes;

7. Fill cell and connect all pressure and monitoring lines;

8. Subject specimen to "conditioning" deviatoric and confining stresses for 15 minutes ($\sigma_1 = 47$ kPa, $\sigma_3 = 21$ kPa and $\sigma_1 = 271$ kPa, $\sigma_3 = 69$ kPa). An approximate haversine pulse of 0.1 s duration at 30 cpm was adopted as described for asphaltic concretes; and

9. Apply combinations of deviatoric and confining stresses to the specimens through the range of desired stress levels (BISTRO analyses of typical pavement structures) and monitor resulting deformations.

A number of granular base and sub-base aggregates used in Southern Ontario were characterized following this approach. Four drainage conditions were considered:unsaturated-drained; unsaturated-undrained; saturated (not 100 percent)-drained; and saturated (not 100 percent)-drained. In addition, both static (constant confining pressure) and dynamic (variable confining pressure) conditions were involved.

Test Results

It was found that the M_R of the base and sub-base specimen, which is highly dependent on confinement, was best given by the relationship (1):

$$M_R = k_1 \theta^{k_2} \qquad (3)$$

where θ is the first stress invariant ($\sigma_1 + 2\sigma_3$ in triaxial tests); and k_1 and k_2 were determined by regression analyses as typically shown in Figure 10. It was also found that stress duration and stress repetitions did not influence the resilient response of granular aggregates. Regression equations for M_R for a number of granular aggregates are summarized in Table 2 showing the high correlation coefficient with θ. Relationships for ν_R were also considered, but appear to be rather complicated and related to principal stress ratio σ_1/σ_3 as shown in Figure 11.

In general, it was found that crushed aggregates were stiffer than pit-run aggregates as indicated in Figure 12. A summary of all test results is given in Table 3, including low gradient permeabilities. These k values have been included to emphasize the importance of considering both stiffness and drainage characteristics during granular aggregate selection. In general, 100 percent crushed aggregates with less than 5 percent 75 µm material are desirable for high quality, free draining base and sub-base construction. Since it was not possible to achieve complete saturation of laboratory prepared specimen, the critical effect of a saturated base under dynamic loading was not shown.

CHARACTERIZATION OF SUBGRADE SOILS

Equipment

In order to characterize the subgrade soil component (fine grained soils) of flexible pavements, the only change from granular aggregates characterization was to prepare specimens using standard Proctor energy at optimum moisture content, and optimum moisture content plus and minus 2 percent. For most of the subgrade soils there was enough cohesion that freezing was not required to place specimens in the triaxial cell.

Test Results

The characterization results for the typical subgrade materials are summarized in Table 4. These results reflect the M_R and ν_R values observed at the lowest and highest stress level applied. It should also be noted that the dynamic condition most closely represents a traffic loading situation.

CONCLUSIONS

For asphaltic concretes, temperature is the most important factor influencing the resilient modulus, with a linear log M_R - temperature relationship demonstrated. Mix type and qualtiy were found to be of secondary influence, but in-service stiffness increases are important and result in M_R values close to those adopted in OPAC. While the resilient Poisson's ratio is usually considered of secondary importance, it is both highly mix quality and temperature dependent. Lower quality mixes have ν_R values approaching 0.5 at higher temperature, which is clearly an indicator of deformation susceptibility

While the detailed results are still being analyzed for use in OPAC, some major conclusions can be drawn from the characterization of granular aggregates and subgrade soils: the most significant factor influencing the resilient response of granular aggregates is the applied state of stress (confining pressure) with the θ model for M_R yielding the highest correlation coefficients; the resilient modulus of granular aggregates increases slightly as the deviatoric stress and principal stress ratio increase; degree of saturation, aggregate type, gradation and density are important secondary factors; higher permeability granular aggregates tend to prevent saturation and consequent strength loss, but this implies a low fines content; and for most applications, the effect of confining pressure on subgrade soils can be neglected, but the resilient properties are highly dependent on the dynamic deviatoric stress.

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SPECIMEN TYPE			TEMPERATURE (°C)	MEASURED ν[1] (POISSON'S RATIO EQUIPMENT)	RESILIENT MODULUS, M_R (MPa)	
					POISSON'S RATIO EQUIP.[2] (DIRECT MEASUREMENT)	DIAMETRAL EQUIPMENT (CALCULATED WITH ν)
SURFACE COURSE MIX	HL-3	EXCELLENT MIX	1.0	0.26	10,600	11,055
			22.3	0.30	1,150	1,390
			44.0	0.44	109	248
		AVERAGE MIX	1.0	0.22	10,400	9,256
			22.3	0.39	1,060	1,080
			44.0	0.45	97	210
		MARGINAL MIX	1.0	0.23	9,400	8,655
			22.3	0.42	1,000	1,120
			44.0	0.50	94	210
BASE COURSE MIX	HL-8	EXCELLENT MIX	1.0	0.27	12,900	18,095
			21.0	0.36	1,850	2,035
			39.0	0.40	250	276
	HL-4	AVERAGE MIX	1.0	0.24	14,100	14,980
			21.0	0.36	1,550	1,360
			39.0	0.42	210	196
		MARGINAL MIX	1.0	0.22	10,200	12,980
			21.0	0.37	1,130	1,310
			39.0	0.45	157	209

[1] Taken from Figure 5 [2] Taken from Figure 6

TABLE 1: COMPARISON OF M_R FOR VARIOUS ASPHALTIC CONCRETE MIXES TESTED IN TRIAXIAL AND DIAMETRAL EQUIPMENT

Site	Specimen	Material Type	Test* Type	$M_R=f(\theta)$	Correlation Coefficient	Standard Error	$M_R=f(\sigma_3)$	Correlation Coefficient	Standard Error
D	SC-4	"A", crushed stone, upper	D	$2.181\ \theta^{.63}$	.981	4.182	$6.699\ \sigma_3^{.617}$	.9012	8.9474
D	SC-8	"A", crushed stone, lower	D	$1.9409\theta^{[illegible]}$	.9882	3.2206	$6.898\ \sigma_3^{.673}$	.9094	8.6186
B	SG-12	"A", natural gravel	D	$1.128\ \theta^{.82}$	.9989	2.2328	$2.176\ \sigma_3^{1.112}$	.9303	9.4168
B	SG-16	"A", natural gravel	D	$1.8238\theta^{.6049}$	.9882	3.7756	$4.0913\sigma_3^{.775}$	.8847	8.6724
B	SG-17	"B", natural gravel	D	$1.618\ \theta^{.6744}$	.9915	2.4346	$5.986\ \sigma_3^{.675}$	.9401	6.2432
B	SG-18	"C", natural gravel	D	$1.734\ \theta^{.676}$	.981	4.4938	$6.389\ \sigma_3^{.64}$	.9074	9.4111
C	SC-19	"A", crushed stone	D	$2.558\ \theta^{.655}$	.9839	3.9052	$7.6589\sigma_3^{.701}$	.8923	10.2394
C	SC-20	"B", crushed stone	D	$3.941\ \theta^{.5893}$	.988	3.7521	$12.836\ \sigma_3^{.587}$	.9131	9.7393
A	SC-G-21	"A", blend	D	$2.8073\theta^{.6193}$	.9891	3.3907	$8.5148\sigma_3^{.6233}$	.9256	8.3658
D	SC-4	"A", crushed stone	S	$2.5379\theta^{.6593}$	.992	3.2957	$7.293\ \sigma_3^{.674}$	.9807	7.0001
D	SC-8	"A", crushed stone	S	$.7352\theta^{.8677}$	.9963	2.155	$3.128\ \sigma_3^{.864}$	.9719	7.3067
B	SG-12	"A", natural gravel	S	$.806\ \theta^{.87}$	.9975	4.6018	$3.648\ \sigma_3^{.864}$	.9674	10.7488
B	SG-16	"A", natural gravel	S	$1.1658\theta^{.805}$	.9965	2.5856	$4.1566\sigma_3^{.8152}$	.975	8.0351
B	SG-17	"B", natural gravel	S	$.948\ \theta^{.82}$	.9987	1.8099	$3.266\ \sigma_3^{.844}$	.9899	4.6374
B	SG-18	"C", natural gravel	S	$.988\ \theta^{.785}$	.9971	1.7966	$2.027\ \sigma_3^{.961}$	.9702	7.9388
C	SC-19	"A", crushed stone	S	$1.132\ \theta^{.824}$	.9966	2.7252	$3.7404\sigma_3^{.8629}$	.9738	8.7116
C	SC-20	"B", crushed stone	S	$1.472\ \theta^{.8012}$	.9886	5.831	$5.3\ \sigma_3^{.812}$	.9665	10.478
A	SC-G-21	"A", blend	S	$1.414\ \theta^{.7943}$	.9976	2.4054	$4.8739\sigma_3^{.811}$	.9774	8.9591

* D = Dynamic S = Static

TABLE 2: REGRESSION EQUATIONS FOR M_R FROM UNSATURATED-DRAINED TESTS ON GRANULAR AGGREGATES

Sample No.	Material	Degree of Saturation Sr %	Specific Gravity Gs	Permeability (i=0.35) cm/sec	CBR %	Modified AASHTO T180 $\gamma_{d(max)}$ kg/m^3	w_{opt} %	Resilient Modulus (MPa)								
								σ_1=83 kPa (12 psi)		σ_3=21 kPa (3 psi)			σ_1=276 kPa (3 psi)		σ_3=69 kPa (10 psi)	
								*	U-D	U-U	S-D	S-U	U-D	U-U	S-D	S-U
SC#4	Crushed Limestone, Upper Level (with natural fines	73	2.85	1.23×10^{-3}	80	2310	6.3	Dy	48.618	52.285	53.584	52.341	95.415	91.214	-	100.795
								St.	56.629	60.252	59.019	63.498	136.142	132.204	130.413	126.788
SC#5	Crushed Limestone, Upper Level (without fines)	78	2.86	9.06×10^{-2}	104	2246	6.3	Dy	45.724	42.979	48.037	48.883	91.598	87.922	95.726	95.726
								St.	57.485	50.123	42.770	50.164	119.041	111.677	130.769	106.630
SC#6	Crushed Limestone, Upper Level (with 5% fines)	77	2.85	5.97×10^{-2}	65	2204	6.0	Dy	39.970	40.919	44.310	44.666	80.021	77.610	86.525	89.470
								St.	48.606	46.561	50.376	49.175	52.019	66.771	78.689	78.599
SC#7	Crushed Limestone, Upper Level (with 10% fines)	72	2.79	1.35×10^{-3}	75	2295	5.4	Dy	47.116	48.853	53.002	52.25	100.77	95.394	105.044	101.111
								St.	61.344	62.410	59.789	61.261	146.834	140.988	147.808	131.049
SC#8	Crushed Limestone; Lower Level (with natural fines)	68	2.85	3.54×10^{-2}	64	2420	5.5	Dy	46.343	46.712	50.313	48.516	101.339	97.485	102.467	93.304
								St.	49.877	50.086	50.542	49.588	130.755	128.943	117.897	110.194
SC#9	Crushed Limestone, Lower Level (without fines)	71	2.86	3.41×10^{-2}	73	2444	5.7	Dy	39.014	36.228	38.666	35.971	85.743	82.016	87.295	80.423
								St.	47.434	42.835	45.489	36.9	134.739	120.030	134.497	100.966
SC#10	Crushed Limestone, Lower Level (with 5% fines)	77	2.85	1.2×10^{-3}	48	2440	5.8	Dy	40.934	42.867	40.583	44.137	88.283	33.325	89.059	90.49
								St.	47.385	53.499	50.432	43.649	126.062	124.827	117.575	104.683
SC#11	Crushed Limestone, Lower Level (with 10% fines)	75	2.85	1.26×10^{-4}	49	2454	5.0	Dy	53.836	45.192	45.767	46.092	80.707	79.492	96.889	86.974
								St.	48.596	44.451	42.835	43.504	116.371	134.600	106.085	106.263
SC#12	Natural Gravel (Semi Crushed Pit Run) (with natural fines)	74	2.79	6.23×10^{-3}	36			Dy	46.891	48.575	56.350	49.784	108.455	103.086	87.618	79.491
								St.	53.556	53.084	49.485	35.338	151.690	136.634	122.474	81.043
SC#13	Natural Gravel (without fines)	78	2.77	8.58×10^{-2}	42	2358	6.6	Dy	42.384	42.271	45.494	43.891	94.121	90.556	94.876	94.876
								St.	49.472	46.643	49.101	44.584	137.164	124.004	141.667	123.453

TABLE 3: SUMMARY OF CHARACTERIZATION RESULTS FOR GRANULAR AGGREGATES

Sample No.	Material	Degree of Saturation Sr %	Specific Gravity Gs	Permeability (i=0.35) k cm/sec	CBR %	Modified AASHTO T180		Resilient Modulus (MPa)								
						γd(max)	w_{opt}	σ_1=83 kPa (12 psi) σ_3=21 kPa (3 psi)					σ_1=276 kPa (40 psi), σ_3=69 kPa (10 psi)			
						kg/m^3	%	*	U-D	U-U	S-D	S-U	U-D	U-U	S-D	S-U
SG#14	Natural Gravel (with 5% fines)	77	2.78	6.8x10^{-3}	38	2331	5.3	Dy	48.993	46.930	51.628	52.283	99.951	97.634	105.919	108.807
								St	57.611	53.633	58.164	47.594	142.163	143.352	144.655	133.900
SG#15	Natural Gravel (with 10% fines)	75	2.77	7.96x10^{-3}	35	2326	5.4	Dy	49.877	55.728	56.528	52.365	118.585	118.001	118.548	107.842
								St	53.056	55.984	56.117	49.383	147.433	142.251	153.729	125.012
SG#16	Natural Gravel (with natural fines)	75	2.78	3.24x10^{-2}	44	2358	6.5	Dy	44.228	43.643	50.065	44.611	92.187	88.393	101.265	92.610
								St	53.341	48.987	56.627	49.619	142.106	127.882	143.697	124.470
SG#17	Natural (with natural fines)	58	2.76	2.8x10^{-4}	42	2098	6.6	Dy	35.069	37.017	44.451	44.867	80.400	75.444	86.542	84.624
								St	44.867	47.325	47.8[illegible]	45.505	123.068	116.919	130.841	124.485
SG#18	Natural Gravel (with natural fines)	80	2.76	4.2x10^{-5}	10	2243	6.8	Dy	41.720	43.910	48.005	50.345	94.54	97.31	101.268	102.473
								St	42.870	46.580	50.726	50.065	127.86	124.19	133.540	124.612
SG#19	Cruhsed Limestone (with natural fines)	70	2.85	3.3x10^{-3}	112	2267	6.7	Dy	50.490	48.060	54.869	56.340	108.080	107.420	108.524	111.450
								St	56.730	54.060	58.674	49.300	150.890	147.680	149.853	123.550
SG#20	Crushed Limestone (with natural fines)	59	2.85	4.8x10^{-3}	132	2337	5.9	Dy	55.017	52.535	58.546	57.152	116.165	111.245	117.258	117.132
								St	68.592	64.079	65.920	55.181	168.968	164.114	161.057	145.400
SG#21	Blend, (Crushed Stone & Natural Gravel Mix)	62	2.87	6.25x10^{-3}	105	2273	7.1	Dy	49.084	48.140	50.008	50.698	103.868	99.371	101.285	99.159
								St	61.977	54.554	57.422	52.182	165.490	148.506	150.320	133.508
** SG#22	Air Cooled Slag	50	2.93	3.0x10^{-3}	-	--	-	Dy	48.303	51.754	56.987	56.315	116.616	115.308	121.952	123.238
								St	66.134	66.794	66.951	63.273	175.100	165.191	170.236	168.548

*Dy - Dynamic Case - Variable Confining Pressure Test
St - Static Case - Constant Confining Pressure Test

U-D --- Unsaturated Drained Test
S-U --- Saturated (Not 100 Percent) Undrained Test

TABLE 3: SUMMARY OF CHARACTERIZATION RESULTS FOR GRANULAR AGGREGATES (CONTINUED)

				RESILIENT MODULUS (kPa)				CBR-100% AASHTO T99		COMPACTION AASHTO T99			
				σ_3=0 kPa		σ_3=20.7 kPa (=3 psi)							
No.	MATERIAL		ω %	σ_1=20.7kPa (=3 psi)	σ_1=41.4kPa (=6 psi)	σ_1=41.4kPa (=6 psi)	σ_1=62.1kPa (=9 psi)	Unsoaked %	Soaked %	$\gamma_{d(max.)}$ kg/m^3	ω(opt.) %	Specific Gravity Gs	Degree of Saturation Sr%
H	Hamilton Steel Cinders	-2% Opt. +2%	8.5 10.5 12.5	79.071 86.975 69.298	105.715 115.241 80.514	80.406 93.183 131.665	98.629 104.230 139.342	42	38	2154.7	10.5	2.91	21.8
B	40%<silt<50%	-2% Opt. +2%	9.4 11.4 13.4	25.071 19.895 17.100	23.757 24.031 24.500	23.150 26.666 39.300	23.697 32.399 36.800	20	2	1811.9	11.4	2.72	20.6
A	Silt<40%	-2% Opt. +2%	7.3 9.3 11.3	10.420 8.433 7.876	13.175 11.190 -	11.817 13.385 9.470	14.004 12.722 -	23	2	2100.2	9.3	2.79	19.5
F	Tobacco Sand	-2% Opt. +2%	12.3 14.3 16.3	11.895 13.973 21.492	- 15.609 23.272	13.033 18.876 62.158	11.429 21.085 59.647	20	18	1674.1	14.3	-	-
C	Silt>50%	-2% Opt. +2%	8.8 10.8 12.8	22.970 18.440 21.500	22.276 21.015 25.400	- 27.885 31.00	- 27.429 34.300	26	2	1887.2	10.8	2.73	20.3
G	Welland Slag	-2% Opt. +2%	- 14.0 16.0	- 26.632 22.409	- 34.944 31.612	- 27.156 21.911	- 35.557 28.637	33	31	1384.1	14.0	2.79	19.5
D	MTC-80-AX-20 Clay	-2% Opt. +2%	- 9.1 -	10.640 28.000 -	9.082 16.400 -	16.580 26.700	11.251 -	8	4	1685.3	21.0	3.0	35.4
E	Leda Clay		σ_1=6.9kPa 4.608		σ_1=20.7kPa 4.922	σ_1=41.4kPa 7.257		2	-	-	-	-	-

TABLE 4: SUMMARY OF CHARACTERIZATION RESULTS FOR SUBGRADES

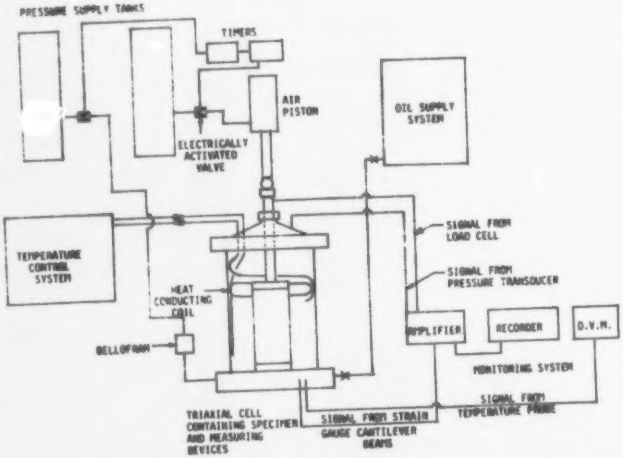
PRESSURE SUPPLY TANKS
TIMERS
AIR PISTON
ELECTRICALLY ACTIVATED VALVE
OIL SUPPLY SYSTEM
SIGNAL FROM LOAD CELL
TEMPERATURE CONTROL SYSTEM
HEAT CONDUCTING COIL
SIGNAL FROM PRESSURE TRANSDUCER
AMPLIFIER
RECORDER
D.V.M.
BELLOFRAM
MONITORING SYSTEM
SIGNAL FROM TEMPERATURE PROBE
TRIAXIAL CELL CONTAINING SPECIMEN AND MEASURING DEVICES
SIGNAL FROM STRAIN GAUGE CANTILEVER BEAMS

FIGURE 1: GENERAL SCHEMATIC OF DYNAMIC TRIAXIAL EQUIPMENT FOR CHARACTERIZING ASPHALT MIXES

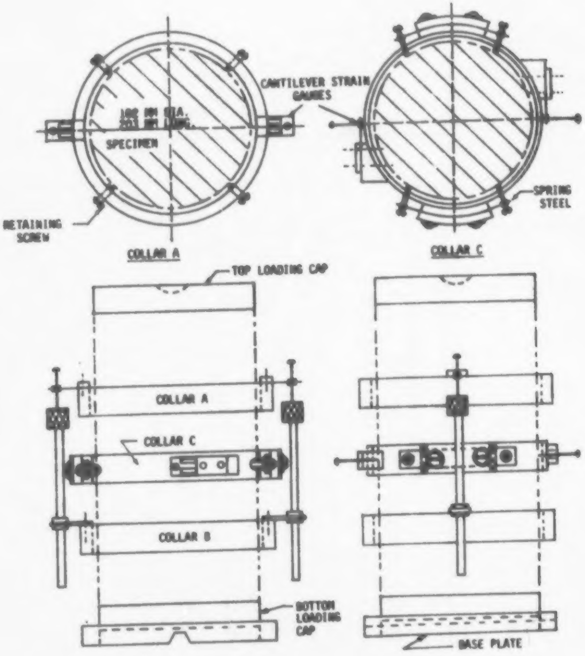
102 MM DIA.
203 MM LONG.
SPECIMEN
CANTILEVER STRAIN GAUGES
RETAINING SCREW
SPRING STEEL
COLLAR A
COLLAR C
TOP LOADING CAP
COLLAR A
COLLAR C
COLLAR B
BOTTOM LOADING CAP
BASE PLATE

FIGURE 2: POISSON'S RATIO MEASURING DEVICE FOR ASPHALTIC CONCRETE

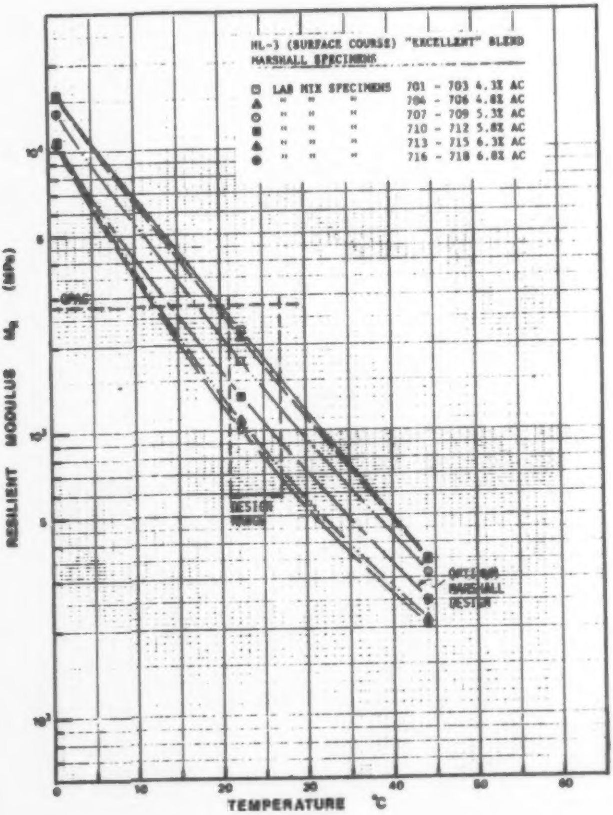

FIGURE 3: RELATIONSHIP BETWEEN LOG M_R AND TEMPERATURE FOR AN EXCELLENT SURFACE COURSE MIX

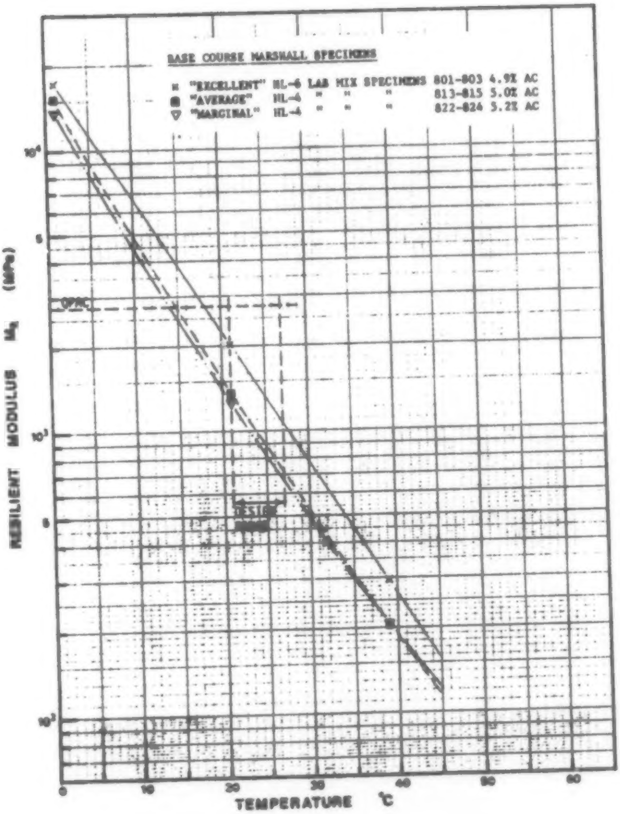

FIGURE 4: COMPARISON OF M_R VALUES FOR EXCELLENT, AVERAGE AND MARGINAL BASECOURSE MIXES

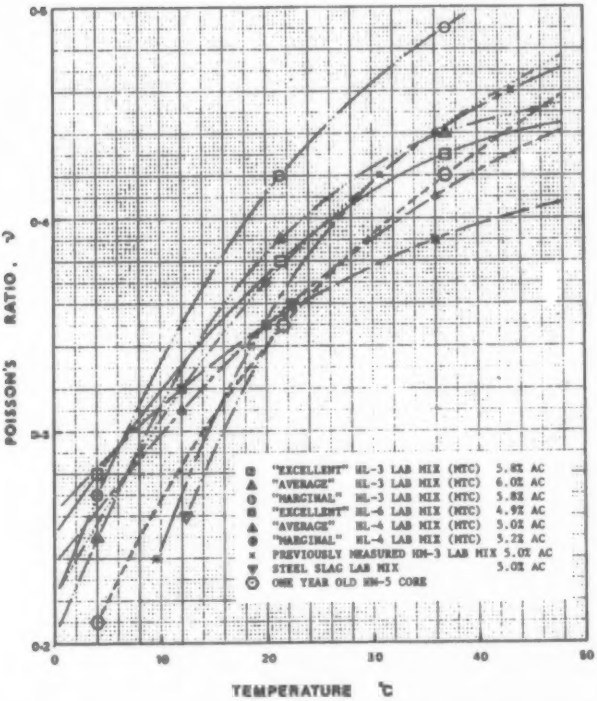

FIGURE 5: RELATIONSHIP BETWEEN ν_p AND TEMPERATURE FOR VARIOUS ASPHALTIC CONCRETE MIXES

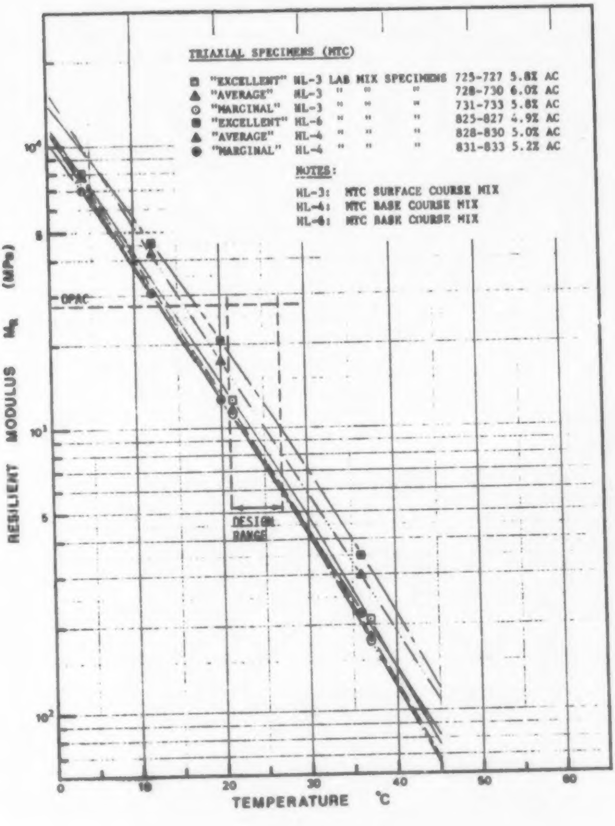

FIGURE 6: RELATIONSHIPS BETWEEN LOG M_R AND TEMPERATURE FOR VARIOUS SURFACE AND BASECOURSE MIXES AT OPTIMUM AC CONTENT

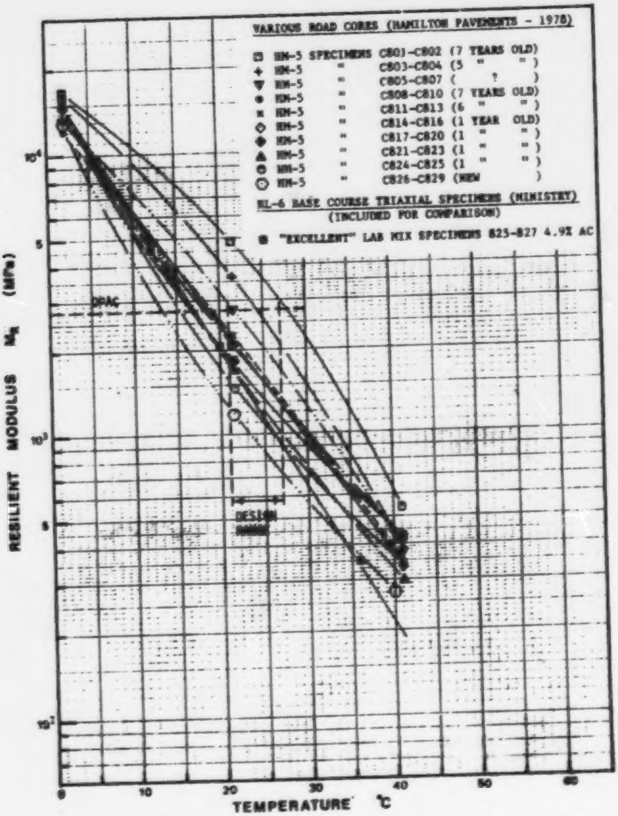

FIGURE 7: RELATIONSHIPS BETWEEN
LOG M_R AND TEMPERATURE
FOR CORES

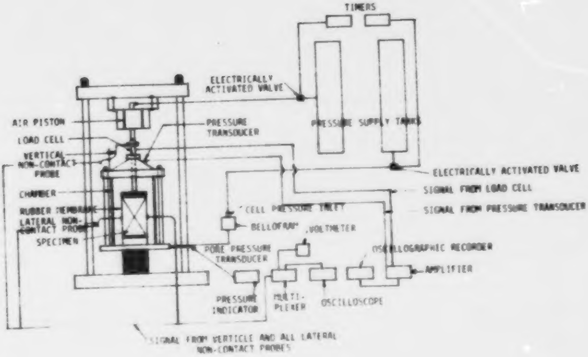
TIMERS
ELECTRICALLY ACTIVATED VALVE
AIR PISTON
PRESSURE TRANSDUCER
PRESSURE SUPPLY TANKS
LOAD CELL
VERTICAL NON-CONTACT PROBE
ELECTRICALLY ACTIVATED VALVE
CHAMBER
SIGNAL FROM LOAD CELL
CELL PRESSURE INLET
SIGNAL FROM PRESSURE TRANSDUCER
RUBBER MEMBRANE
LATERAL NON-CONTACT PROBE
BELLOFRAM
VOLTMETER
SPECIMEN
PORE PRESSURE TRANSDUCER
OSCILLOGRAPHIC RECORDER
AMPLIFIER
PRESSURE INDICATOR
MULTI-PLEXER
OSCILLOSCOPE
SIGNAL FROM VERTICLE AND ALL LATERAL NON-CONTACT PROBES

FIGURE 8: SCHEMATIC OF DYNAMIC TRIAXIAL RESILIENT MODULUS EQUIPMENT FOR GRANULAR AGGREGATES AND SUBGRADE SOILS

FIGURE 9: PHOTOGRAPH OF TRIAXIAL CELL WITH NON-CONTACT PROBES

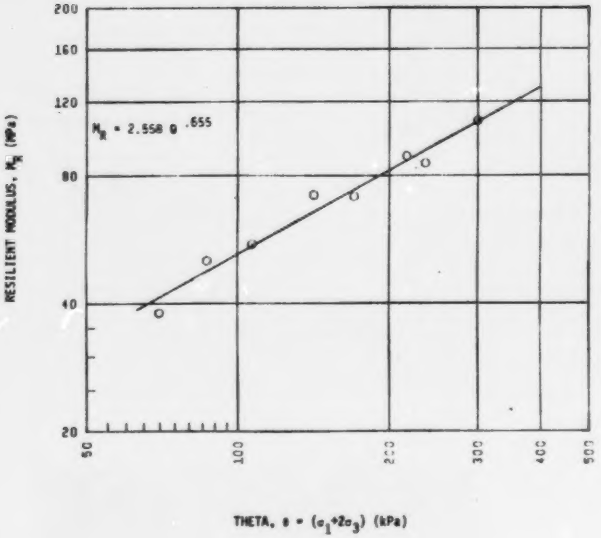

FIGURE 10: M_R AS A FUNCTION OF Θ,
CRUSHED LIMESTONE

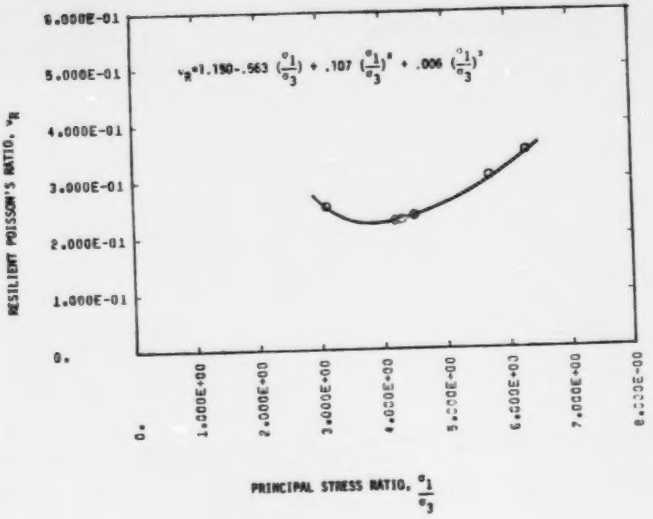

FIGURE 11: ν_R AS A FUNCTION OF PRINCIPAL STRESS RATIO, CRUSHED LIMESTONE

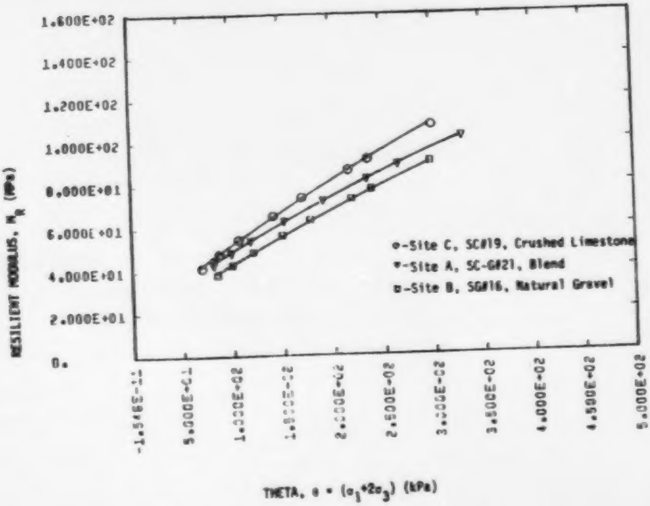

FIGURE 12: M_R RESULTS FOR DIFFERENT TYPES OF GRANULAR 'A' AGGREGATES

Session 5

Reinforcement of Asphalt Pavements with a High Strength, Plastic Geogrid

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REINFORCEMENT OF ASPHALT PAVEMENTS WITH A HIGH STRENGTH, PLASTIC GEOGRID

by
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ABSTRACT

The notion of reinforcing flexible pavements has existed for some years, and a few attempts have been made to use metallic and other materials. These have not been cost and performance effective.

Recently, however, a new, high strength plastic geogrid material known as Tensar has become available and pavement reinforcement was suggested as one of its possible civil engineering applications. Consequently, the first phase of a research program initiated in early 1981 examined the potential of a variety of materials for pavement reinforcement, including Tensar. The conclusion was that this new material did indeed offer potential and should be further evaluated.

An initial material testing program involving polyethylene Tensar was carried out in 1981. The purpose was to determine the stress-strain characteristics of Tensar, for use in predicting its behaviour under varying temperature and loading conditions.

A comprehensive experimental program of "model" tests of reinforced and unreinforced pavements was then carried out in the latter half of 1981 and early 1982. This paper describes the experimental program and the comparative results that were obtained.

Variables in the experiment were different thicknesses of full depth asphalt, reinforced vs. unreinforced sections, and different subgrade

strengths. Dynamic, repeated loads to simulate loaded truck traffic were applied and measurements were made of cumulative permanent deformation, static rebound, fatigue cracking, etc. As well, instrumentation was installed to measure dynamic and static tensile strains in the Tensar mesh itself, at the bottom of the asphalt layer and vertical stress in the subgrade on one series of tests.

The results showed that the Tensar mesh was very effective as a reinforcement, in terms of carrying double the number of load repetitions, giving thickness savings between 50 and 100 mm, and minimizing fatigue cracking. Because of this effectiveness, it seems well worthwhile as a next step to develop a field installation/construction technology and carry out full-scale field trials.

1. INTRODUCTION

1.1 Background

Many existing roads are becoming inadequate due to rapid growth in traffic volumes and axle loadings. Escalating costs of materials and energy, as well as a lack of resources to finance new roads and rehabilitate existing ones, provide an incentive for the exploration of alternatives. Flexible pavement reinforcement is one such alternative. If it could reduce the thickness of paving materials and/or extend pavement life in a cost and performance effective way, then it would certainly be a viable alternative.

The major reasons that attempts to reinforce flexible pavements have been almost non-existent include the following:

1. Asphalt prices until about the mid-1970's were sufficiently low to preclude any economic viability of reinforcing these pavements;
2. The only suitable material, steel, was far too expensive in relation to any thickness savings that might occur; moreover, steel rusts very quickly in an asphalt pavement;
3. Non-metallic materials, such as fabrics, have been used to a significant degree in some areas of North America and have sometimes been claimed to have reinforcement properties, among other attributes. However, their low strength, high extensibility and low modulus makes this doubtful. Moreover, there is little if any documented evidence to demonstrate that any fabric reinforces a pavement or extends its life, notwithstanding that some fabrics may be effective in a crack reflection and/or waterproofing role in more temperate climates.

However, asphalt prices, aggregate prices and energy costs have increased dramatically in recent years. These factors, combined with new technological developments in non-metallic reinforcing materials, suggest that the feasibility of reinforcing flexible asphalt pavements may be worthwhile.

1.2 Feasibility Study of Flexible Pavement Reinforcement

In late 1980, a comprehensive research program was initiated in the subject area, with the first phase being an evaluation of existing metallic and non-metallic materials, including geotextiles (1). It also included the design and carrying out of an experimental program as a cooperative effort between Royal Military College (RMC) at Kingston, the Ontario Ministry of Transportation and Communications, Gulf Canada Ltd. and the University of Waterloo.

The primary candidate arising from the evaluation was a new high strength, plastic mesh or geogrid material known as TENSAR. This material, subsequently described in more detail, is made from polypropylene and is biaxially oriented to give strengths in the order of mild steel in both directions. Material testing of Tensar was performed at the University of Waterloo.

1.3 The Experimental Program and Its Objectives

The experimental program was carried out at RMC in Kingston. The design involved varying thicknesses of reinforced and unreinforced full depth asphalt with varying strengths of subgrade in a 2.4 m x 4 m test pit, as subsequently described in more detail.

For each test series, half the pavement was reinforced with TENSAR mesh while the other half was left as a control section for comparison. The main goal of the experimental program was to thoroughly investigate, under a variety of conditions, the mechanical behavior and load-carrying capabilities of flexible pavements which had been reinforced with TENSAR mesh. When compared with unreinforced (control) sections, the structural benefits of the reinforcement could be evaluated. The results of the experimental program were intended to accomplish the following:

1. Examine the mechanical behavior and performance of TENSAR reinforced asphalt pavements under static and dynamic loading;

2. Compare this behavior and performance with that of similar unreinforced sections loaded in an identical manner;
3. Establish an equivalency between depth of TENSAR reinforced and unreinforced asphalt pavement.

As well, the results of the program were to be used to verify and/or modify the basic elastic layer theory and develop initial design procedures for reinforced asphalt pavements. This aspect is not reported herein. It will be covered in subsequent reports arising from the research.

1.4 Objectives of the Paper

This paper describes the development of the reinforced flexible pavement concept, including the design analysis involved and the experimental program which was carried out to verify the concept.

More specifically, the objectives of the paper are as follows:

1. To describe the reinforced pavement concept in terms of the basic mechanisms involved;
2. To describe the experimental program with respect to test facility, variables included, materials, test procedures, series of tests conducted, etc;
3. To present the major test results showing comparisons between reinforced and unreinforced sections, with respect to dynamic strains, load applications carried to "failure", static deflections, permanent deformation, etc.

2. REINFORCED PAVEMENT CONCEPT

2.1 Review of Reinforced Pavement

There are many applications of construction techniques which use reinforcement elements strong in tension or bending to enhance the strength and stability of other materials; for example, using steel bars in reinforced concrete when it is subjected to high tensile stresses or strains.

The term "reinforced pavements" refers to the use of one or more of the pavement layers. Pavement reinforcement has generally been associated, however, with Portland Cement Concrete (PCC) pavements. Here the purpose of the steel reinforcement is to hold cracks together and thereby reduce maintenance and extend life.

Another application of pavement reinforcement is the use of reinforcement elements in concrete overlays to prevent reflection cracking [2, 3, 4, 5]. As for the use of reinforcement elements in asphalt pavements to provide an adequate tensile strength to the asphalt layer, very few trials have been made [6, 7, 8]. In most of these investigations models and field trials were carried out to investigate the effect of using steel or fabric materials to improve the tensile properties of the asphalt layer [9, 10]. These approaches, however, have the following deficiencies.

1. The empirical rules derived from observations and measurements made on these models and the full scale structures provide little information about the fundamental mechanical behavior of the pavements and the reinforcement elements.
2. The extension of such an empirical approach to different types and layouts of reinforcement in different pavements would be difficult because of the trial and error development involved in the approach.

A better approach would be to assess and analyze the behaviour of a reinforced pavement and use this as a basis for design, through the consideration of the following factors:

1. The properties of the reinforcement;
2. The mechanical behaviour of the pavements under various conditions of traffic and environmental stresses;
3. An understanding of the basic mechanisms which operate when reinforcement is incorporated in the pavement.

2.2 The Initial Analytical Investigation as a Basis for the Experimental Program

One of the first steps in this investigation was to carry out a fundamental study of the effect of the major variables involved. This was accomplished through an analytical phase, involving the following (1): a) evaluation of available structural theories, selection of the most appropriate one and modifications if necessary, (b) application of the selected theory to a range of possible design situations for reinforced pavement structures, (c) identification of critical stresses and strains and best location(s) for the reinforcement, and (d) compatability of the reinforcement material with asphalt concrete.

The results were used to plan an experimental program. This program was intended to verify the basic hypothesis (i.e., that flexible pavements could be effectively reinforced), to provide feedback for subsequent updating and/or modification of the analysis and to provide a basis for planning and carrying out full-scale field trials.

The following sections present details of the experimental part of the investigation.

3. THE EXPERIMENTAL INVESTIGATION

3.1 Test Facility

Experimental pavement sections were constructed in a test pit at the Royal Military College (RMC) in Kingston. A brief description of the facility follows.

(a) Test Pit

Asphalt pavement sections comprising TENSAR reinforced and unreinforced control sections were constructed within a 4 m by 2.4 m by 2 m deep concrete pit (Figure 1). The test pit includes a sump and water distribution system at the bottom which allows the pit to be flooded to any desired depth.

(b) MTS Loading System/Data Acquisition

The MTS Loading System at the RMC Structural Engineering Laboratory comprises three independent closed-loop electro-hydraulic actuators and ancillaries designed and packaged by MTS (Material Testing Systems, Co. Ltd.). For this investigation a hydraulic actuator rated at 50 kN (11.25 kips) was used (Figure 2). Each actuator is capable of generating compressive and/or tensile loads in either static or dynamic modes and is controlled through a PDP 11/34 computer. Program instruction is in BASIC and input/output can be made through a CRT console or hard copy terminal (Figure 3).

The versatility of the MTS loading system is reflected in its ability to produce programmed loading conditions ranging from a simple static case to a complex impact loading situation. The PDP 11/34 can also be programmed to read acuator load and displacement at any point in the loading history, reduce the data and output the results in tabular or graphical form.

3.2 Controlled Variables in the Experimental Program

3.2.1 Test Loops

The experimental program was divided into five series of tests, called loops. Each loop presented one set-up of the test pit, in which half of the

Figure 1/ Test Pit with Compacted 1 m Depth of Sand Subgrade in Place

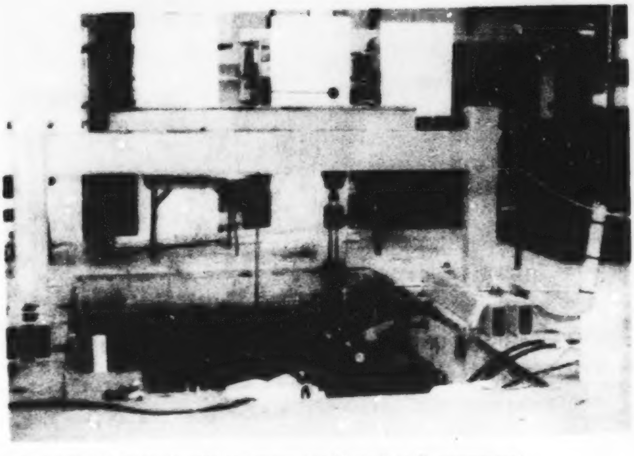


Figure 2/ MTS Hydraulic Actuator and Loading Arrangement

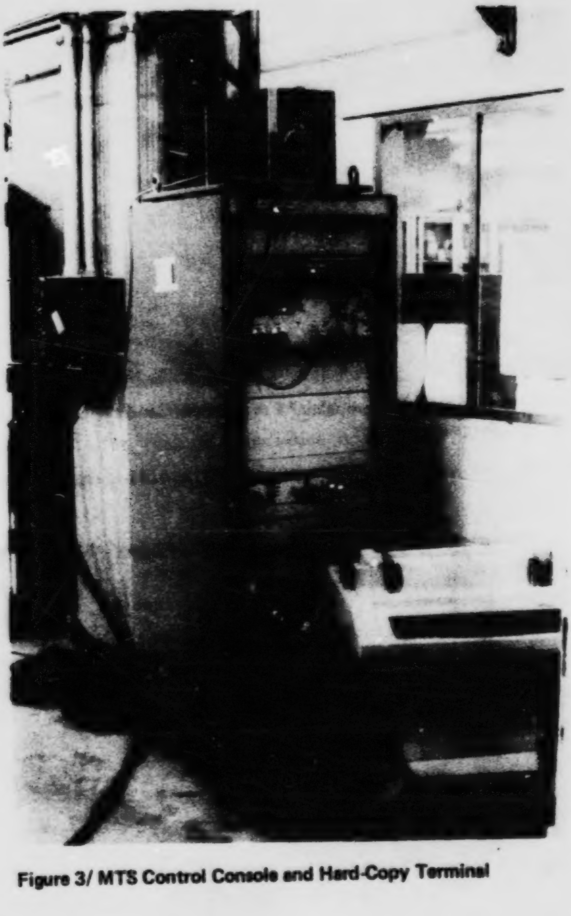


Figure 3/ MTS Control Console and Hard-Copy Terminal pit was reinforced with Tensar mesh and the other half was left as a control section. For each loop, the asphalt thickness and the subgrade condition (either dry or saturated) were the controlled variables. Between 4 and 9 tests were performed for each loop, with each test representing a different location within the test pit. The five loops are described in Table 1.

3.2.2 Objectives of Test Loops and Testing Sequence

The main objectives of the test loops of the experimental program was to determine and compare the TENSAR reinforcing effect in the asphalt pavement. The design of these test loops was conducted in a logical manner to evaluate and examine certain parameters related to reinforcement evaluation. For example, the first loop was designed to compare the behavior and performance of the reinforced section with unreinforced section with the same thickness (150 mm) on a weak subgrade. Permanent deformation and vertical deflections were monitored through the test until complete failure occurred on the unreinforced section.

In loop 2, strain carriers were installed at the bottom of the asphalt layer to compare tensile strain, in the critical zone, between reinforced section and unreinforced sections of the same thickness (165 mm) on a stronger subgrade. Results of these two loops were of major importance since they compare the reinforced sections with unreinforced sections under identical geometric, loading and environmental conditions.

Upon achievement of the first objective (basic comparisons between reinforced and unreinforced), the second objective of finding the equivalent thickness of the TENSAR reinforced layer had to be determined. Loops 3 and 4 were planned and carried out to establish a relationship between the thicknesses of the reinforced and unreinforced sections. In loop 3, two unreinforced sections (200 mm and 250 mm) were tested against a thinner reinforced section of 153 mm on weak subgrade. Results of this loop showed that a value of (50-100 mm) equivalent thickness may represent the TENSAR reinforcement effect.

Based on this finding, loop 4 was carried out with an unreinforced section of 250 mm and a reinforced section of 200 mm to confirm the minimum saving value (50 mm).

TABLE 1

TEST LOOPS AND CONTROLLED VARIABLES

Loop No.	Test No.	Asphalt Thickness	Subgrade Condition	Description
(1)	1	150 mm	Dry	Control
	2	150 mm	Dry	Reinforced
	3	150 mm	Saturated	Reinforced
	4	150 mm	Saturated	Control
(2)	1	165 mm	Dry	Control
	2	165 mm	Dry	Reinforced
	3	165 mm	Dry	Reinforced
	4	165 mm	Dry	Control
(3)	1	250 mm	Saturated	Control
	2	150 mm	Saturated	Reinforced
	3	150 mm	Saturated	Reinforced
	4	200 mm	Saturated	Control
	5	250 mm	Saturated	Control
	6	150 mm	Saturated	Reinforced
(4)	1	200 mm	Dry	Reinforced
	2	200 mm	Dry	Reinforced
	3	250 mm	Dry	Control
	4	250 mm	Dry	Control
	5	250 mm	Saturated	Control
	6	250 mm	Saturated	Control
	7	200 mm	Saturated	Reinforced
	8	200 mm	Saturated	Reinforced
	9	200 mm	Saturated	Reinforced
(5)	1	115 mm	Dry	Control
	2	115 mm	Dry	Control
	3	115 mm	Dry	Reinforced
	4	115 mm	Dry	Reinforced

The last loop was designed to compare the vertical stresses on the subgrade (strong subgrade) for reinforced and unreinforced asphalt sections of the same thickness (115 mm).

Results of the five loops are subsequently presented and discussed in the paper.

3.2.3 Dynamic and Static Loading Patterns

Tensar reinforced and unreinforced pavement sections were loaded through a 300 mm (12 in.) diameter rigid circular plate placed on the pavement surface. The loading pulse was sinusoidal, with an amplitude or peak of 40 kN for each cycle, and a frequency of 10 Hz. The loading program was designed to represent typical traffic loadings on pavements under operating conditions. The cyclic loading was carried out until certain defined criteria for failure, as subsequently described, were reached.

After certain numbers of selected cycles, dynamic loading was discontinued and static loading sequence [5-10 static cycles] applied as a time lengthened, step-wise approximation of one cycle of loading. In addition to obtaining static load response per se, this static loading sequence was necessary for monitoring the array of displacement gauges, strain gauges and strain carriers in each section. During static loading, 10 kN increments of load were applied, to a maximum of 40 kN, followed by smaller decrement to 0 kN. For loops 4 and 5 this was changed into a single increment and single decrement (0-40-0 kN). Dynamic loading was then resumed.

3.3 Test Materials and Preparation of Test

3.3.1 Subgrade Preparation

The subgrade for each loop consisted of a 1.2 m depth of medium to coarse sand. A grain size distribution curve for the subgrade material is given in Table 2. The sand was initially placed in 150 mm lifts and compacted at an optimum moisture content of 11.5 percent using a Whacker VGP 160 Plate tamper. Prior to each subsequent loop, the original subgrade condition was duplicated by removing the top 150 mm of sand, turning over the next 150 mm, and recompacting in two lifts at optimum moisture content. Moisture content and compactive effort were carefully controlled using a Troxler nuclear densitometer.

TABLE 2

SIEVE ANALYSIS OF THE SUBGRADE

Sieve Number	% of Total Passing
3/8"	100
4	95
8	88
16	73
30	50
50	20
100	5
200	-

For selected tests the subgrade was flooded from below to the sand-asphalt interface in order to fully saturate the sand. The compacted subgrade prior to asphalt placement is shown in Figure 1.

3.3.2 Asphalt Layer Construction

For the first 4 loops a local MTC grade HL4 hot mix asphalt was used. A 25 mm lift of hot mix (125° C) was first placed on the subgrade for all tests. The Tensar mesh was then placed on half the pit, the other half being left unreinforced, as the control section. Next, reinforced and unreinforced halves were covered with one additional 50 mm of asphalt and compacted using four passes of a plate tamper. Additional uniform 25 mm to 75 mm lifts were placed and compacted to bring the pavement structure to the desired thickness. The density of the pavement was monitored using a Troxler denistometer to ensure uniform asphalt consistency between reinforced and unreinforced test sections. Samples of the asphalt mix were taken from each loop and were analysed at the Gulf Canada Research Centre. Table 3 shows results of this analysis.

3.3.3 TENSAR Geogrid

The Tensar Geogrid material is a 50 mm by 50 mm mesh made by stretching a sheet of polypropylene plastic with holes punched through it, in both length and width directions. The resultant mesh has "strands" which are about 1 mm thick by 3 mm wide at their narrowest, and "nodes" at junction of strands which are about 10 mm square and about 2 mm thick. The material weighs about 0.210 kg/m^2, is supplied in 3 m by 50 m rolls, and is estimated to cost about \$2/m^2. Other lighter weights and smaller mesh dimensions of

TABLE 3

ASPHALT MIX TEST RESULTS

A) Asphalt Mix Properties

Property	Loop 1 (Aug. 81)	Loop 2 (Sept. 81)	Loop 3 (Oct. 81)	Loop 4 (Nov. 81)	Loop 5 (Jan. 82)
% Asphalt	5.05	5.60	5.70	5.44	5.50
Stability	2765	2248	2470	N/A	N/A*
Voids	2.8	6.6	4.8	N/A	N/A
Flow	10.75	10.75	10.75	N/A	N/A
Corr. Stability	2767	2141	2388	N/A	N/A
V.M.A.	14.7	19.2	17.9	N/A	N/A

*N/A - Not Available

B) Sieve Analysis

Mesh	Loop 1	Loop 2	Loop 3	Loop 4	Loop 5
1-1/2"	100.0	100.0	100.0	100.0	100.0
3/4"	100.0	100.0	100.0	100.0	100.0
5/8"	100.0	100.0	100.0	100.0	100.0
1/2"	94.1	100.0	98.1	93.2	100.0
3/8"	77.1	94.7	81.2	77.3	88.2
4	51.7	77.9	67.3	53.2	63.1
8	41.8	67.6	57.5	41.5	52.1
16	35.9	58.0	50.5	34.3	43.5
30	30.7	48.7	45.6	29.4	32.7
50	21.6	29.4	32.6	19.5	14.1
100	9.1	9.7	13.5	8.3	3.6
200	3.8	4.4	4.6	3.3	1.0
Dust	0	0	0	0	0

geogrid are available. It has been termed as a "Geogrid" to differentiate it from the conventional geotextile fabrics. "Geogrid" describes the open mesh structure which allows interlocking with surrounding materials and thereby mobilizes its high tensile strength.

The basic geometry and physical and mechanical properties of TENSAR, as provided by the manufacturer are shown in Table 4.

TABLE 4

PROPERTIES OF TENSAR*

Structure	Polymer Grid
Width	1.0 m to 3.5 m
Weight	137 gm/m^2 to 729 gm/m^2
Axial Structure	Both Uniaxial and Biaxial
Tensile Strength	$17.0x10^4$ to $56.0x10^4$ K Pa
Modulus in Tension	@ 4,500 MPa
Extension 1/2	< 25%
Thermal Stability	-60°C up to 80°C
Chemical Resistance	To all natural occurring alkaline and acidic concentrations
Biological Resistance	To attack by bacteria, fungi and vermin

*From NETLON Ltd., Kelly Street, Blackburn, England

3.3.4 Low-Temperature Characterization of Tensar

One of the first major questions which arose about Tensar was its low temperature behavior. As a consequence, a testing program was conducted at the University of Waterloo in 1981. The general purpose was to assess its stress-strain-time-temperature response and assess whether the material behaved in a viscoelastic manner, like asphalt. If so, this behavior should be capable of being characterized through the principle of time-temperature superposition.

Direct tensile testing for time-temperature superposition was performed in 1981 at the University of Waterloo using the Tensar SR product. This is an oriented, high density polyethylene (HDPE) mesh. The results obtained from these tests are not directly applicable to the polypropylene Tensar because of the latter's higher melting point but other properties are similar between the two polymers (11).

This means that the time-temperature superposition principle should apply to polypropylene in the same way as to HDPE. Some experimental work will be required to obtain the appropriate graphs and values for use in predicting the behaviour of the polypropylene Tensar.

A Wykeham Farrance testing frame with adjustable crosshead speeds was fitted with special jaws that clamp securely onto the sample during the tests. For both high and low temperature tests, an insulated environmental chamber encased the sample and the jaws. The sample was cooled either by vapour drawn off from liquid nitrogen, or heating was done with a hot air blower for tests in which temperatures higher or lower than room temperature were required. Figure 4 shows a sample of TENSAR SR1 clamped within the chamber.

Tests were performed at -40°C, -10°C, 20°C and 70°C, at rates of strain ranging between 0.0034% per minute to 4.28% per minute. The samples were each measured for length and cross sectional area, then submitted to a constant rate of extension, at the required temperature, with the load and strain being recorded at regular time, t, from the beginning of each test. A more complete description of the testing apparatus and methods is contained in Ref. (12).

The experimental data acquired was used to set up a graphical means of predicting the stiffness modulus at any temperature and rate of strain within the ranges tested.

The experimental results are plotted on one log-log graph with stiffness modulus, (t x stress/% strain) as a function of t, where t is a specific time from the beginning of the test, with the stress and % strain of the sample measured at that time. This transformation causes all of the tests, performed at the same temperature to align onto one curve. Figure 5 shows a series of curves for the temperature tested. The master curve was produced by shifting the temperature curves horizontally until they aligned on an arbitrary reference temperature which in this case was chosen as 20°C. The number of log decades that each curve was shifted is plotted as a_T verses temperature on Figure 6.

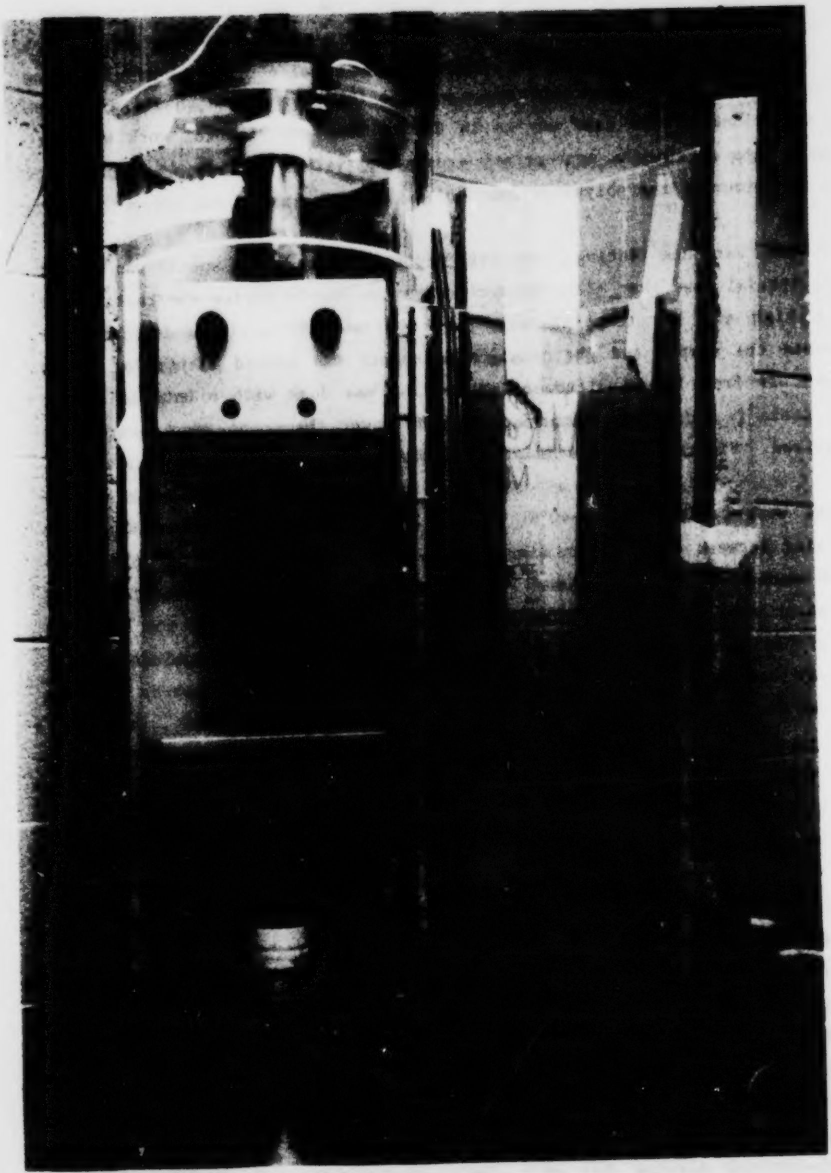


Figure 4/ Environmental Testing Chamber

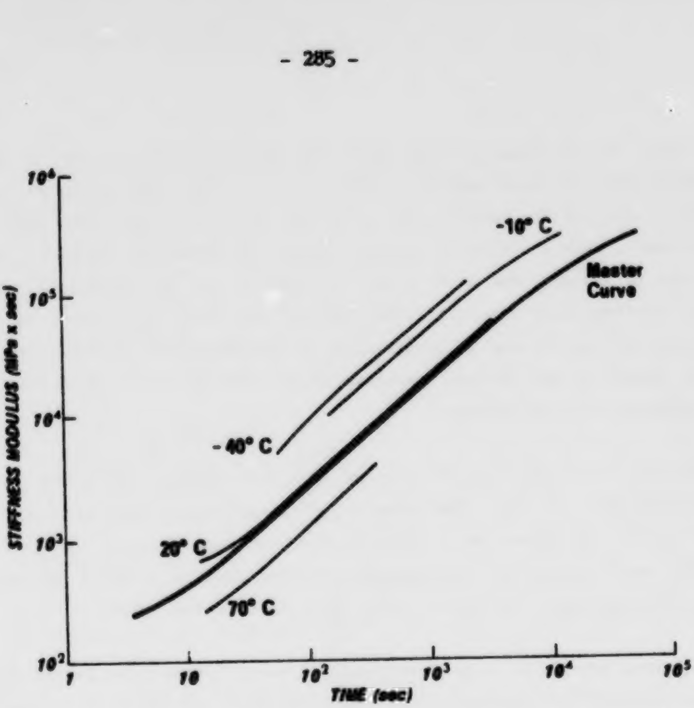


Figure 5/ Temperature Curves and Master Curve

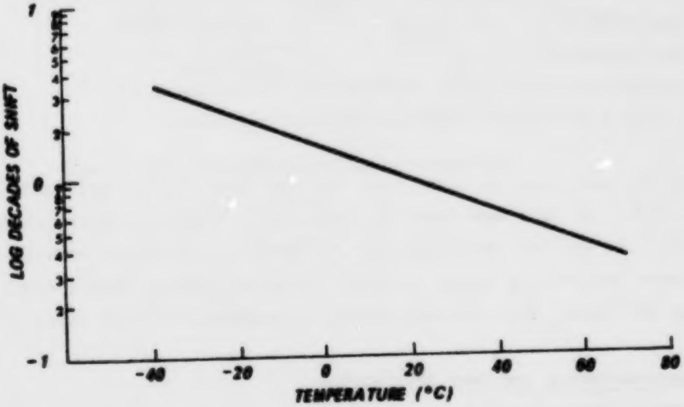


Figure 6/ Shift Factor, a_T

This shift factor graph, along with the master curve can now be used to determine the stiffness modulus of Tensar SR1 at any temperature and rate of strain. The shift factor, a_T , yields the curve for the appropriate temperature, with a stiffness modulus associated with each time, t. Knowing the rate of strain, the % of strain at each t can be calculated, and the stress derived from the stiffness modulus at each point. In this way, stress-strain curves may be constructed to determine the strength and behavior of Tensar at any desired temperature or rate of strain corresponding to the particular installation.

Traffic loadings apply strain very quickly, but strains due to low temperatures will occur slowly. The behavior of Tensar under both of these conditions can be predicted with time-temperature superposition as long as a suitably wide range has been included in the testing which provides the basis for developing the master curve and shift factor curve.

These graphs are still to be developed for the polypropylene Tensar AR1 used in the pavement reinforcement, but it is expected the basic principles shown to be applicable to the SR1 product can also be used for the AR1.

The Tensar type used in this research is designated as AR-1, and it, as with other Tensar products, has been developed to have the following:

1. High tensile strength
2. High modulus
3. Low elongation
4. Biaxial structure (i.e., strength in both directions)
5. A grid with desired width for pavement purposes

The Tensar mesh used in each loop was cut from a roll provided by Gulf Canada Ltd. On each mesh used in loops 1, 2, 3 and 5, strain gauges were bonded to the top and the bottom of the TENSAR at locations covering a wide area under the loading plate, in order to monitor the strains induced in the mesh by the loads. This instrumentation is subsequently described.

3.4 Instrumentation and Data Acquisition

The general arrangement of instrumentation used to monitor pavement performance is shown in Figure 7. The instrumentation and recording devices are summarized in the following sub-sections.

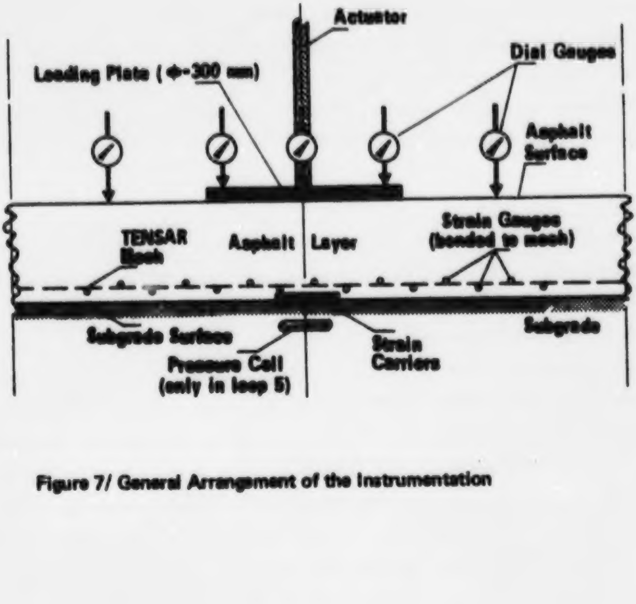

Actuator
Dial Gauges
Loading Plate (Φ-300 mm)
Asphalt Surface
Strain Gauges (bonded to mesh)
TENSAR Mesh
Asphalt Layer
Subgrade Surface
Pressure Cell (only in loop 5)
Strain Carriers
Subgrade

Figure 7/ General Arrangement of the Instrumentation

3.4.1 Actuator Load Cell and LVDT

The actuator load cell and LVDT (Linear Variable Displacement Transducer) are part of the closed-loop, electro-hydraulic control system for the MTS actuator. During static load cycles, both devices were accessed through the PDP 11/34 computer at pre-programmed intervals to record plate load and displacement (13).

3.4.2 Strain Gauges

Foil-type (120 ohm) strain gauges (SHOWA Y11-FA-S-120) were bonded to the top and bottom of the TENSAR mesh (Figure 8a) at locations directly below the loading plate and at radial distances from the load centre. The strain gauges were used to record the magnitude and distribution of elastic and plastic tensile strains generated in the reinforcement elements as a result of the loading. The strains were read during static load cycles using 10-channel Vishay Strain Indicator/Balance Boxes with quarter-bridge completion circuitry. For the last loop (5), the strain gauges were monitored during dynamic load cycles using a multi-channel recorder and magnetic tape supplied by MTC.

3.4.3 Strain Carriers

Mastic "strain carriers" were supplied by the Alberta Research Council. Each unit consists of two (120 ohm) strain gauges embedded in a 150 mm square by 12 mm thick mastic plate. The mechanical properties of the mastic are compatible with those of the asphalt concrete. For each test set-up a strain carrier was placed directly under the centerline of the loading plate and at the subgrade-asphalt interface (Figure 8b). This location corresponds to the zone of anticipated maximum tensile strains in the asphalt layer. The effectiveness of the TENSAR mesh as a reinforcing element for reducing these tensile strains can be determined by comparing the response of strain carriers in the reinforced and control test sections. The strain carrier output was recorded in the same manner as the strain gauges.

3.4.4 Dial Gauges

Dial gauges were located on the rigid loading plate (140 mm from the centre) and at radial distances from the plate. The dial gauges were read during static load cycles and were used to determine the elastic and plastic surface deflection profile with the number of load cycles.

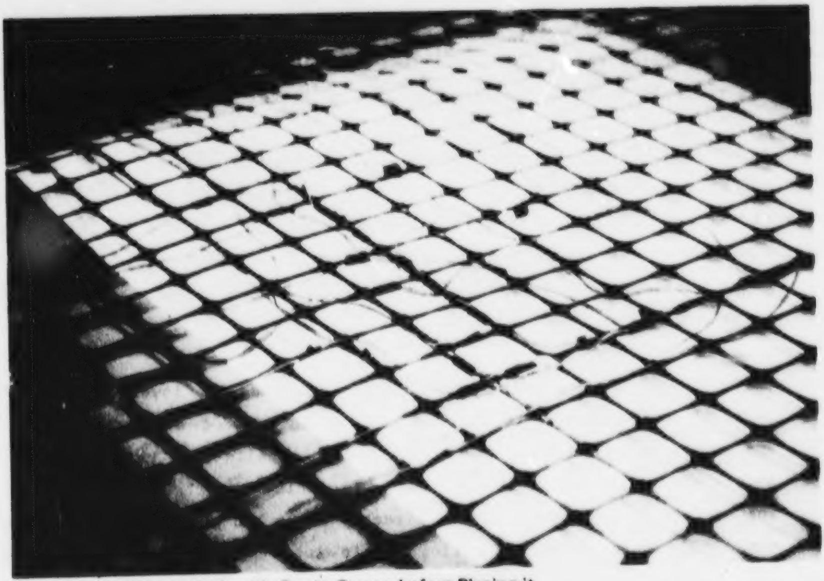


Figure 8a/ TENSAR Mesh with Strain Gauges before Placing it in Asphalt

Figure 8b/ Locating Asphalt Strain Carriers

3.4.5 Pressure Cells

For loop 5, a circular plate pressure cell was embedded in the subgrade directly below the centre of loading for each test set-up. Depth of burial was about 50 mm below the sand/asphalt interface.

Each pressure cell consists of two cylindrical aluminum plates, 133 mm in diameter fastened back to back to give a total thickness of 13 mm. The plates are in the form of a recessed disc made up of a 6.5 mm thick annulus surrounding a pressure sensitive diaphragm 2 mm thick. A four-arm strain gauge configuration (Wheatstone Bridge) is bonded to each top and bottom diaphragm of the pressure cells.

The pressure cells were placed in the subgrade to investigate relative vertical stress gradients generated below the loaded areas for reinforced and unreinforced test sections. The pressure cells were calibrated in situ before and after the final test loop by placing the rigid loading plate at the surface of the sand subgrade directly above the pressure cell. The pressure cells were monitored during static load cycles using a multichannel data acquisition supplied by RMC.

3.5 Failure Criteria

In order to objectively compare the performance of the reinforced and control sections, certain failure criteria have been established. Failure was said to have occurred if:

1. A permanent deformation of 30 mm was measured;
2. The development of extensive cracks, or
3. A steady increase in the measured values of stress, surface deflection and/or horizontal strain at the interface.

Table 5 summarizes these failure criteria. The values were applied to both Control and Reinforced sections in all loops in order to determine the number of load cycles at which failure actually occurred. It was clear from the observations and the test results that a number of factors affected the mode of failure for each loop. Among these factors are the age of the asphalt when the test begins, the temperature and the subgrade (dry or saturated).

TABLE 5

FAILURE CRITERIA

(1)	Total Permanent Deformation	30 mm
(2)	Extensive Cracks	
(3)	Deflection	20% increase
(4)	Strains	30% increase
(5)	Stresses on the Subgrade	steady increase

It is noteworthy that the failure of the TENSAR mesh did not have to be considered in the criteria. The reason is that the strains on the mesh on all loops did not exceed 30% of its yield strain of 15%.

4. RESULTS

4.1 Results of the First Loop

Results of the first loop showed that TENSAR reinforcement had improved the permanent deformation resistance of the reinforced pavement and extended the pavement life dramatically. Figure 9 shows the measured permanent deformation against number of cycles for two reinforced and unreinforced sections tested in loop 1. Up to 50,000 cycles, there was little difference in rutting. It was with increasing cycles that the permanent deformation difference became dramatic.

A factor of some importance is that loop 1 was a "debug" loop, and because insufficient time was given for curing (i.e., only 24 hours from placement to starting the testing), the mix was somewhat soft.

It is also important to note that the unreinforced section had failed completely at the end of the test (punched through) while the reinforced section remained together. This latter observation suggests that lower level of tensile strain and vertical stresses were provided by the reinforcement layer. This observation was to be confirmed in the next test loop.

4.2 Results of the Second Loop

Similar to loop 1, this loop was carried out to compare the performance of control and reinforced sections for the same pavement thickness on a strong subgrade. As seen in Figures 10, 11 and 12, the following results may be noted.

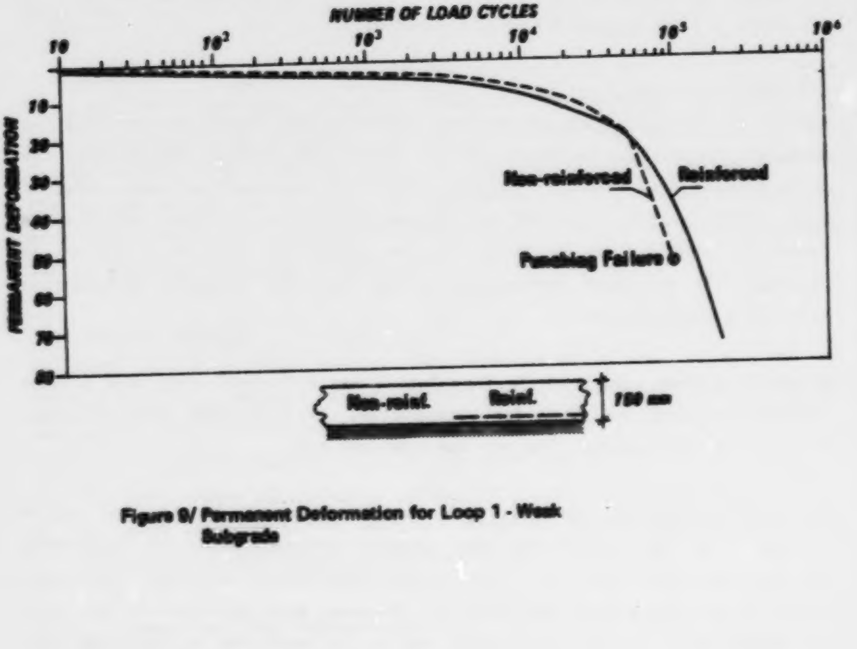


Figure 9/ Permanent Deformation for Loop 1 - Weak Subgrade

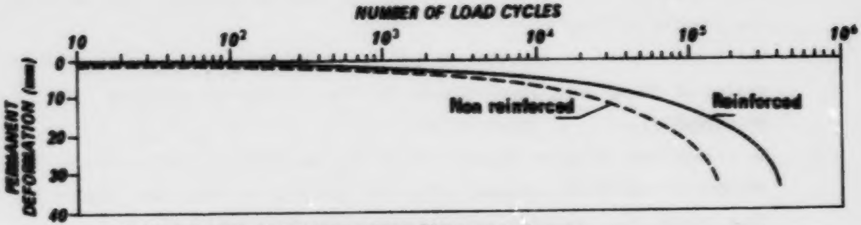


Figure 10/ Permanent Deformation for Loop 2 - Strong Subgrade

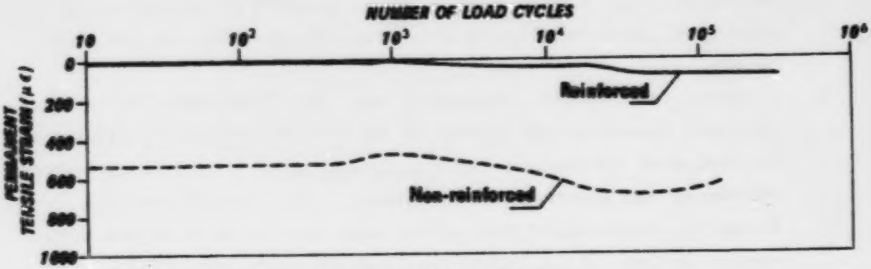


Figure 11/ Permanent Tensile Strain in the Asphalt at Bottom of the Layer for Loop 2

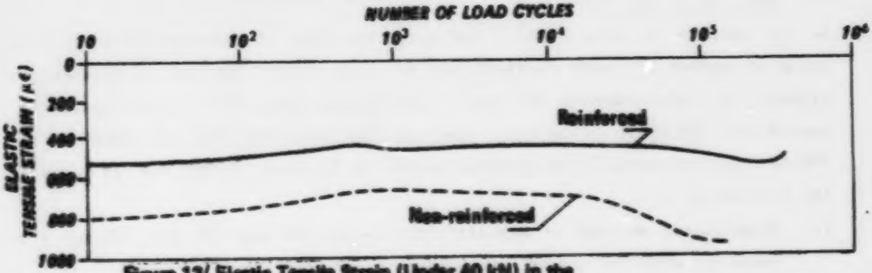
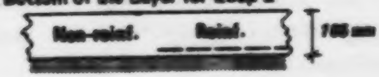


Figure 12/ Elastic Tensile Strain (Under 40 kN) in the Asphalt at Bottom of the Layer for Loop 2



1. The total penetration (permanent deformation) for the control section was higher at the end of the test compared to the deformation of the reinforced section at the same number of loading cycles. This confirms the results of loop 1.
2. The total number of repetitions of the 40 kN load to a limiting deformation of 30 mm more than doubled for the reinforced sections (350,000 vs. 150,000).
3. No significant plastic tensile strain was measured at the bottom of the reinforced sections compared with the control section (see Figure 8). This result confirmed the explanation given for loop 1 of the low level of stresses and strains induced in the reinforced sections.
4. The value of the elastic tensile strain measured at the bottom of the reinforced layer was less by more than 30% than that for the control section.
5. A highly significant observation was the development of tension (fatigue) cracks on the surface of the control sections (initiated at the bottom of the asphalt layer). Clearly, the reinforcement reduces the number and severity of such cracks. This observation also confirmed the reasoning of the failure mode that occurred in the control section in loop 1.

4.3 Results of the Third Loop

As the results of loop 1 and 2 indicate, the life of pavement sections, in terms of number of loads carried, can be doubled for the same thickness of asphalt, if reinforcement is used. Loop 3 was designed to investigate the equivalent thickness of asphalt that may be saved due to the presence of TENSAR reinforcement. The results shown in Figures 13, 14 and 15 suggest the following.

1. Significant savings of asphalt thicknesses (between 50 and 100 mm) are possible with the use of reinforcement (Figure 13).
2. No significant plastic strains were monitored in the case of the thinner (150 mm) reinforced sections, but they were significant on the unreinforced sections (Figure 14).
3. The measured vertical elastic rebound for the thinner reinforced section was smaller than the thicker (200 mm) control section and slightly larger than that of the (250 mm) control section (Figure 15). This is another indicator that additional structural strength is added to the thinner section due to the presence of the reinforcement.

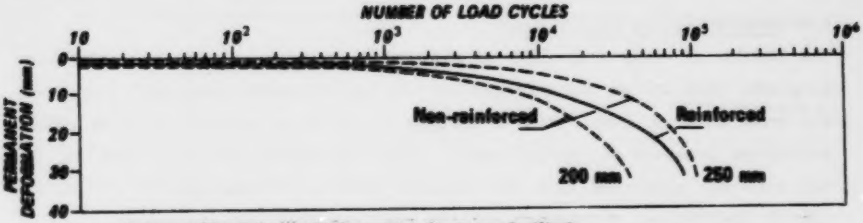


Figure 13/ Permanent Deformation for Loop 3 - Weak Subgrade

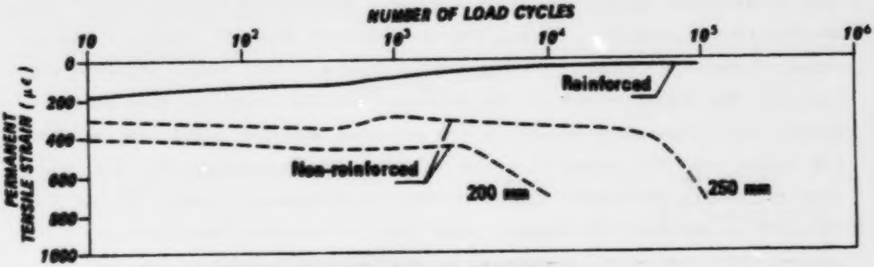


Figure 14/ Permanent Tensile Strain in the Asphalt at Bottom of the Layer for Loop 3

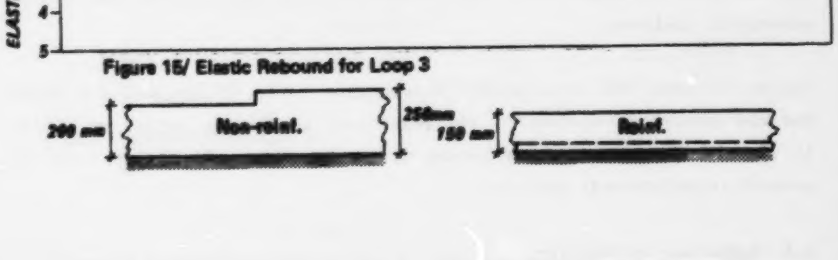
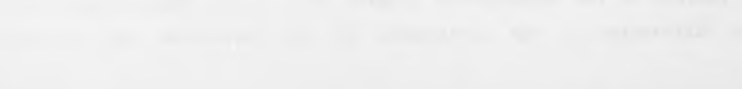


Figure 15/ Elastic Rebound for Loop 3


200 mm
Non-reinf.
250mm
150 mm
Reinf.

The results of this test loop suggest that a minimum saving of asphalt thickness of 50 mm is possible.

4.4 Results of the Fourth Loop

The results of this loop (200 mm and 250 mm reinforced, dry and saturated subgrade, with 250 mm unreinforced and dry and saturated subgrade) confirmed the hypothesis of loop 3 that a minimum of 50 mm of asphalt could be saved due to the presence of reinforcement. Also, as can be seen in Figure 16, it was strongly confirmed that the elastic tensile strains due to the 40 kN load in the thinner reinforced section are less than the strains on the (50 mm) thicker control section. Furthermore, Figure 16 shows two more significant differences between the tensile strains measured on the reinforced and unreinforced pavements. First, the unreinforced pavement showed a higher value of residual strain or creep (εp in Figure 16 b1) which explains the cause of the higher values of the permanent tensile strain on this section. Second, the compressive strain on the unloaded adjacent reinforced section was higher than that measured on the unloaded adjacent control section under similar conditions (almost double). This difference is likely due to the interaction between the asphalt layer and the reinforcement geogrid. Of course, this adds to the structural strength of the reinforced pavement.

4.5 Results of the Fifth Loop

The last loop featured pressure cells under each test section to monitor the vertical stresses 50 mm under the subgrade interface. The presence of these cells (aluminum plates, 155 mm diameter) affects the values of the permanent deformation because they act as additional reinforcement for the subgrade layer. However, the major objective of this loop was to monitor the vertical stresses under the loaded sections and to use the data in subsequent analyses.

Figure 17 shows the relationship between the ratio of the measured stress for the control of section to the reinforced section vs number of cycles. It clearly shows that the subgrade stress is 30 to 40% higher under the control (unreinforced) section.

4.6 Comparison of Results

The results of the experimental program have clearly demonstrated significant differences in the performance of the reinforced and unreinforced

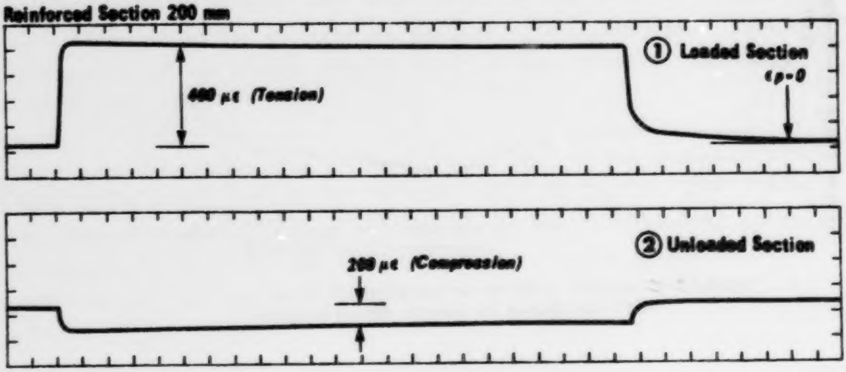


Figure 16a

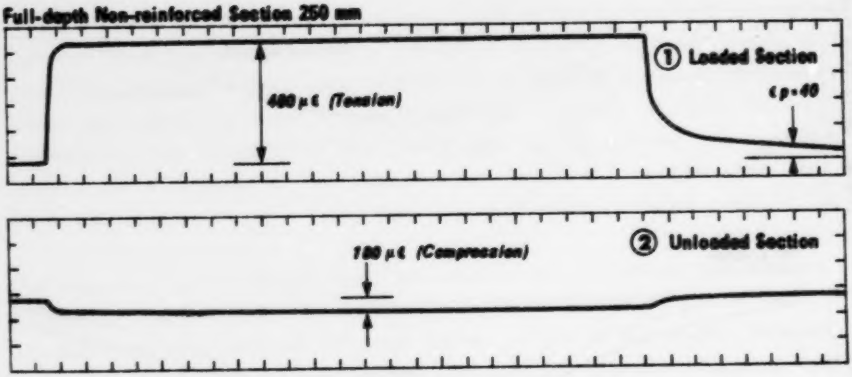


Figure 16b

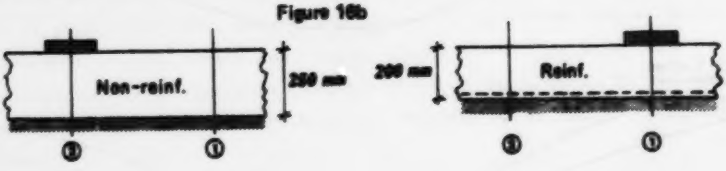

Non-reinf.
250 mm
②
①
200 mm
Reinf.
②
①

Figure 16/ Tensile Strain Pulses for Loop 4 - Weak Subgrade

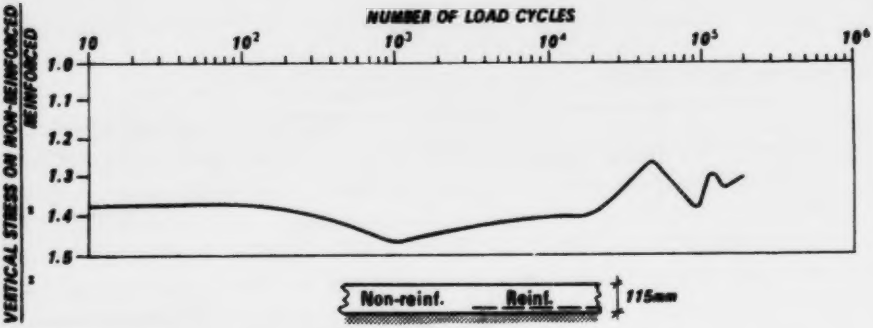


Figure 17/ Stress Ratio of the Unreinforced Section to the Reinforced Section for Loop 5 - Strong Subgrade

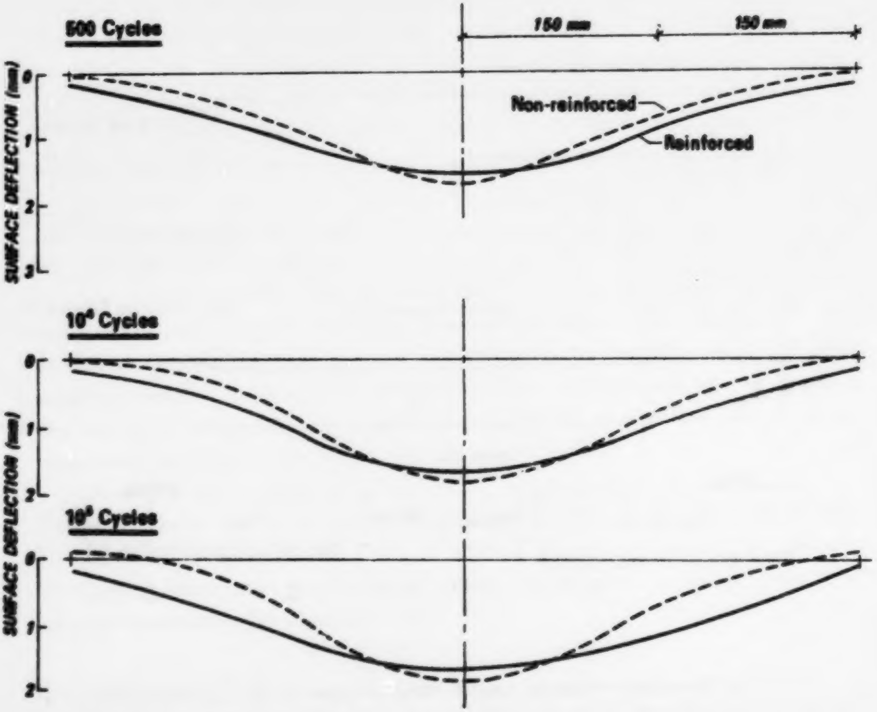


Figure 18/ Surface Deflection Bowls of Unreinforced and Reinforced Sections for Loop 2 sections. Results of Loops 1 and 2 showed that reinforced sections of the same thicknesses would carry more than double the number of load cycles to failure compared to the unreinforced sections. The reinforcement reduces the magnitude of the elastic tensile strain at the bottom of the asphalt layer by about 30%.

Cracks were observed early on the unreinforced sections and progressed into severe cracks on the surface. On the other hand, very few (hairline) cracks were observed on the reinforced sections at the end of the loading cycles. Two unreinforced sections, one in each loop, were punched through while the reinforced sections were holding together even after higher numbers of load cycles.

Results of loops 3 and 4 showed that possible savings between 50 to 100 mm of asphalt material will be obtained by reinforcing the thinner sections. In loop 3, the comparison between the 150 mm reinforced and 200 mm unreinforced showed that the thinner reinforced section is significantly stronger than the unreinforced section. The results of this loop suggested that a 50 mm savings with reinforced section will result in less permanent deformation, less permanent tensile strain and higher number of load cycles. Furthermore, the presence of the TENSAR mesh in the thinner sections of loops 3 and 4 resulted in the reduction of the elastic tensile strain by about 20% compared to the strains under the thicker unreinforced layer.

The comparison of the 150 mm reinforced section with the 250 mm unreinforced section of loop 3 showed that the maximum possible savings of asphalt material under the circumstances is about 100 mm. Although the elastic rebound of the 250 mm unreinforced section was less than the elastic rebound of the 150 mm reinforced section, the reinforced section performed better as far as the permanent tensile strain and fatigue cracks were concerned. This latter observation raises an important question about which criteria is more imporotant in pavement design; is it permanent deformation or fatigue cracking or deflection?

The results of loop 5 helped in explaining the findings of the previous loops. As shown in Figure 19, the ratio between the vertical stresses on the subgrade under the unreinforced sections were 30-40% higher than the stresses under the reinforced sections. This difference is explained by the effect of the reinforcing mesh. The vertical stresses will be distributed on a larger area under the reinforced section, resulting in less pressure on the subgrade. This difference can be further explained by Figure 18. In this figure, deflection bowls at different load cycles for reinforced and unreinforced section in loop 2, are shown. In this figure, the following differences may be realized.

1. The spread of the deflection bowl for the reinforced section is larger than for the unreinforced.
2. The slope of deflection bowls for the unreinforced is steeper than the slope of the deflection bowls for the reinforced section suggesting higher values of shear and vertical stresses.
3. Fatigue cracks developed on the unreinforced section resulted in smaller area of stress distribution and higher stress values (10^5 cycles).

Table 6 summarizes some selected results of the 5 loops carried out through the experimental program.

TABLE 6

SUMMARY OF SELECTED RESULTS

Loop No.	Section	Thickness mm	Total No. of Cycles	Total Penetration mm	Permanent Tensile Strain με
1	Control	150	100,000	53.0*	
	Reinforced	150	100,000	37.0	
			200,000	76.0	
2	Control	165	150,000	30.0	760
	Reinforced	165	350,000	27.1	140
3	Control	255	113,500	31.8	735
	Control	200	37,500	29.3	700
	Reinforced	150	90,000	29.7	00
4	Control	255	135,000	15.0	710
	Reinforced	200	135,000	13.0	180
5	Control	115	300,000	10.2	880
	Reinforced	115	300,000	6.8	80

*This section punched through at 100,000 cycles

5. CONCLUSIONS

This paper presents a new, effective method of reinforcing asphalt pavements using a new non-metallic high tensile strength geogrid. It also presents a methodology for instrumenting, constructing and testing reinforced pavements under loading conditions close to those of actual road conditions.

The results described in the paper clearly illustrate the structural strength which was provided to the asphalt layer by reinforcement, and describes a method to predict strength changes due to varying temperatures and loading conditions. It also provides a simple tool to compare the performance of the reinforced sections with unreinforced sections under different conditions.

With the world wide emphasis on energy and conservation of resources, reinforced flexible pavements offer a promising alternative to conventional designs. The potential of this new technique can be summarized as follows:

1. a thickness saving of up to 100 mm of asphalt material;
2. up to double the number of load repetitions;
3. prevention or minimization of fatigue cracks in the asphalt layer;
4. less maintenance due to the reduced rutting and cracks;
5. possibility of higher axle load limits.

This potential of reinforced flexible pavements suggests that the development of a field installation/construction technology and full-scale field trials would be a worthwhile and next logical step.

ACKNOWLEDGEMENTS

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Session 6

Modified Asphalts in Japan and Research into the Development of New Rubberized Asphalts

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MODIFIED ASPHALTS IN JAPAN AND RESEARCH INTO THE DEVELOPMENT OF NEW RUBBERIZED ASPHALTS

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ABSTRACT

In Japan, various kinds of modified asphalts are used for the special requirements of pavement construction, such as resistance to plastic flow and wearing resistance.

But the modified asphalts have become insufficient to prevent plastic flow of pavement in warm areas and wearing of pavement in cold areas because of the increase in traffic and the use of large vehicles in recent years.

This paper describes the present state of modified asphalts in Japan and research on the development of new rubberized asphalt to meet the requirements of resistance to plastic flow and to wearing on recent heavy traffic roads.

INTRODUCTION

The rapid development of the economy in Japan has resulted in larger-sized vehicles and an increase in traffic volume. The roads are in service under very severe conditions. In addition, new requirements for roads necessitate the raising of quality standards for asphalt pavement, particularly with respect to service conditions for roads having flow or wearing of the asphalt pavement. In any case, damage to the pavement cannot be left as it is from the viewpoint of traffic safety, noise, and vibration. Therefore, more durable asphalt pavements having higher service quality are required.

In Japan, mix design factors such as the grading of mixture or the designed asphalt quantity are usually altered to suit the desired anti-flow or anti-wear qualities of the pavement. However, recently, modified asphalts are used as special measures.

Asphalt has a tendency to flow at high temperatures and to be fragile at low temperatures due to its high temperature susceptibility. In order to improve this property, research on rubberized asphalt has been conducted at the Ministry of Construction, universities, and various private research organizations, and has received diverse evaluations, resulting in an increasing demand for rubberized asphalt every year. In the meantime, a modified asphalt with a thermoplastic resin was examined and, presently, various types of modified asphalts are marketed. Although those modified asphalts have produced satisfactory results under suitable service conditions, their performance, especially on the heavy traffic roads, is not always satisfactory. The development of an asphalt material with better qualities is expected.

In this report the current position of modified asphalts in Japan and the rubberized asphalts under development as anti-flow and anti-wearing measures in heavy traffic roads are described.

1/ THE CURRENT POSITION OF MODIFIED ASPHALT

1.1/ Outline

The current position of modified asphalts shall be described with reference to literature and data published so far in Japan.

1.2/ The Background of Modified Asphalts

The first appearance of rubberized asphalt pavement in Japan was the use of NR (Natural Rubber) powder by the Tokyo Metropolitan Office in 1952. In the same year Kobe City used scrapped rubber and in 1954 the Hokkaido

Development Bureau used NR powder in the construction of National Highway route 36.

In 1960, the Natural Rubber Research Institute developed the NR/asphalt master batching method and the premix method was merchandised. After that, as the domestic petrochemical industry developed, the synthetic rubber SBR (Styrene Butadiene Rubber) became widely used.

The plant-mix method was first executed in 1965 in Sapporo City by means of the imported SBR latex, and in the next year it was used in the construction of National Highway route 36 by the Hokkaido Development Bureau. The demand for this method has grown rapidly because of the simple and rational production of the rubberized asphalt mixture and by the appearance of a domestic SBR latex (used in 1967 on National Highway route 12). In recent years, as shown in Table 1.1, 100,000 to 150,000 t (converted amount to asphalt) came to be used annually as a binder for anti-wearing, anti-flow, and anti-skid pavements. Apart from this, at present, various types of modified asphalts with thermoplastic resin or thermoplastic elastomer added are used as binders for anti-flow and anti-wearing pavements. The Ministry of Construction has been conducting research into a semi-blown asphalt which has an improved thermo-susceptibility and higher viscosity at 60°C, by not adding asphalt modifier but by a blowing operation, for use as an anti-flow measure for heavy traffic roads. The semi-blown asphalt test was made on roads directly controlled by the Ministry of Construction. The results show a good service quality in terms of anti-flow [2]. However, it is now being re-examined from the anti-crack aspect by altering the 60°C viscosity standards from the current 14,000 ± 4,000 poise to 10,000 ± 2,000 poise.

1.3/ Uses of Modified Asphalt

The modified asphalt is used as one of the countermeasures for pavement wearing caused by studded tires or tire chains in snowy and cold areas and for flow in warm areas.

Many of these results were reported. In general, they show certain achievements but cannot become an effective measure in actual situations. The main reason for this is the variety of the properties of modified asphalts and the unclearness of the binder because the service property in actual roads is affected by many factors such as the design of the asphalt mixture, the execution conditions, the undergrade and base course conditions, and by environmental conditions like sunshine.

1.4/ Variety of Modified Asphalts

Modified asphalts currently marketed for anti-flow and anti-wearing may be classified as shown in Table 1.2. Normally, a high polymer is used as a modifier. Table 1.3 provides an overview of the modifiers either currently being used or under examination.

1.5/ Quality Standards for Modified Asphalts

The quality standard for semi-blown asphalt with the higher 60°C viscosity and the standard property of the premixed and plant-mixed rubberized asphalts are set out in the Asphalt Pavement Manual [5]. A rubber added (Type I) and a rubber and thermoplastic resin added (Type II) are prescribed [6], fitting the standard for the modified asphalt in the bridge surface pavement of long bridges where long term durability is expected.

Quality standards for these modified asphalts are shown in Table 2.2. Property objectives of developed binder are shown in Section 2 (excepting semi-blown asphalt).

1.6/ Properties of Modified Asphalts

Properties of modified asphalts vary greatly depending upon the types and quality of asphalt, upon the amounts of modifier added, and according to the manufacturing method. As an example, types of rubber and properties of rubberized asphalt are shown in Table 1.4 and the properties of rubberized asphalt according to different types of raw asphalt are shown in Table 1.5.

Furthermore, one of the property test results of modified asphalts now marketed is shown in Table 1.6. The comparison result of properties of marketed modified asphalts indicates that most of them have about the same penetration as straight asphalts, but their softening points and 60°C viscosity figures, which are indicators for anit-flow property, range greatly from around 80/100, equivalent to the figure of straight asphalt, to AC-80, equivalent to that of blown-asphalt.

The toughness tenacity which is an indicator for cohesion and tenacity of aggregates ranges from a figure which does not reach that for the rubberized asphalt standard property in the Asphalt Pavement Manual, to a figure which is two to three times higher.

The 180°C and 120°C viscosities that are indicators of the difficulty of production mixing and of compaction at paving, are higher than that of straight asphalt, and resin-added asphalts tend to have a higher figure than rubberized ones.

In conclusion it is clear that modified asphalts cannot be treated uniformly.

1.7/ Mixture Properties of Modified Asphalts

1.7.1/ Anti-Flow Property

Typical test results are shown in Figures 1.1 and 1.2 which indicate how the modifying effects of asphalt affect the anti-flow property in the pavement mixture. Figure 1.1 shows that the effect of adding SBR is significant in the dynamic stability of the wheel-tracking test. Figure 1.2 indicates that the dynamic stability of the wheel-tracking test varies greatly depending on the type of modified asphalt. Rubberized asphalt showed little effect but asphalts with resin added showed a great effect.

1.7.2/ Anti-Wearing Property

Typical test results are shown in Figures 1.3 to 1.5 which indicate how the modifying effects of asphalt affected the anti-wearing property in the pavement mixture. Figures 1.3 and 1.4 show the results of the ravelling test. The results vary greatly depending on the type of modified asphalt: there are some with fairly good improvement while there are others with no effect. These results suggest that the curve may be affected by the properties of the base asphalt as well as of the type of high polymer added. Figure 1.5 shows the results of the improvement upon the fragility at low temperatures. The point of becoming fragile (the temperature of the highest bending strength) was affected little by adding SBR. However, it showed that the bending strength in the fragile area was affected.

1.8/ Service Properties of Modified Asphalts

In order to evaluate the service properties of modified asphalts in actual pavements, various tests were conducted. Typical experimental pavement results are shown in Figures 1.6 and 1.7. Figure 1.6 is the result from Okabe's experimental pavement over eight years. Rubberized asphalt showed less rutting due to the modifying effect of rubber. Figure 1.7 shows the results from Kohjunai's experimental pavement over three years. Progress of rutting in winter (wearing) varies greatly depending on the type of modified asphalt and added resin and part of the rubberized asphalt seemed to have a certain effect. The effect of modified asphalt on the rutting in summer (flow) is not always clear.

Some other tests show an effect of rubberized or resin-added asphalt, but others show none. Practical capabilities of modified asphalt cannot be uniformly evaluated.

1.9/ Conclusion

The modifying effect of modified asphalts varies depending on the type of high polymer added and the type of base asphalt. Modified asphalts show various properties and it is necessary to develop modified asphalts having a higher quality and to use the right material in the right place according to the objectives of the pavement, such as anti-flow pavement or anti-wearing pavement, in order to meet the higher requirements for pavements in recent years.

Furthermore, modified asphalts that have high polymers, such as rubber, added generally have a high viscosity. Therefore, there is difficulty in mixing or in compaction. It is necessary to execute the work properly in order to maximize potential and to establish suitable construction techniques to utilize the properties of these new pavement materials.

2/ DEVELOPMENT OF RUBBERIZED ASPHALT

2.1/ The Outline of Research

Modified asphalt is used to improve anti-flow or anti-wearing serviceability in asphalt pavements. However, with the recent heavy traffic roads, the serviceability of modified asphalt is not necessarily sufficient. Therefore, the development of higher-quality modified asphalts is desired.

Research was conducted in order to clarify the properties of modified asphalt currently being used for pavements, and to develop a binder with better anti-flow and anti-wearing qualities. Development objectives of the binder for the service properties of anti-flow and anti-wearing are set in Table 2.1. From the aspect of the material properties of the binder, the material property objective was set in Table 2.2 assuming the standard properties of rubberized asphalt in the Asphalt Pavement Manual, the modified-asphalt standards of Bridge Surface Pavement between Honshu and Shikoku and other survey information. The service property objective is achievable.

As described in the Table 2.3, this research began with a survey of literature and data to grasp the status quo of technology concerning the characteristics and service properties of modified asphalt. Then, properties of conventional modified asphalts were clarified by material property tests of premix-type, modified asphalts currently being marketed. After this, based on these results, the development objectives and a trial manufacturing method of the binder to be developed will be defined. After material property tests of various trial manufactures, potential binders for development will be selected. Further, a mixing test will be applied to the selected binders and the binder which can show superior characteristics will be chosen. Then, the applicability to the site shall be examined by serviceability tests. According to these results, a quality standard and site application manual shall be clarified.

This research is to be conducted over three years, from 1981 to 1983. This report covers the trial manufacture and material property test of the binder to be developed from the tests concerning asphalt binders done in the first year of the research. The research flow chart is shown in Table 2.3.

2.2/ Test Samples

The selection of raw asphalt materials and an asphalt modifier for the trial manufacture of potential binders was done in the following way.

2.2.1/ Selection of Raw Material Asphalt

The selection of raw asphalt material was done by referring to the manufacturing output of one refinery of an oil company in the penetration grades of 40 to 60, 60 to 80, and 80 to 100. The top five manufacturers, according to consumed quantity, were selected. However, for grades with penetrations of 40 to 60, only four manufacturers were available.

2.2.2/ Selection of Asphalt Modifier

After the survey of literature, rubber was selected as the asphalt modifier. SBR (Synthetic rubber) and NR (Natural rubber) were selected as representative rubbers. The basic material properties of these modifiers are described in Table 2.4.

2.3/ Trial Manufacturing Conditions of the Binder to be Developed

A trial manufacturing condition of the binder to be developed was conducted under the conditions in Table 2.5a by using the indoor manufacturing equipment shown in Figure 2.1.

2.4/ Testing Method

Tests concerning the potential binders were conducted in the order of a raw material asphalt selection test, a trial manufacture and material property test.

2.4.1/ The Raw Material Asphalt Selection Test

A general material property test and a chemical property test were conducted as a raw asphalt material selection test.

2.4.1.1/ General Material Property Test

Table 2.5b describes the test combinations and the contents of the test. The general material property test was conducted on asphalt of grade 3 penetration from five manufacturers, and the modifying effect on a trial manufactured binder of a 3% concentration of SBR (moonie value 50) was evaluated.

2.4.1.2/ Chemical Property Test

A chemical analysis was made of the properties of raw asphalt material, especially the composition analysis and the molecular weight distribution from the aspect of mutual solubility with rubber.

The composition analysis method of asphalt was done by the Iijima method in Figure 2.2 using the fraction method by solvents. The measuring of the molecular weight distribution was done by the GPC method (Gel Permeation Chromatography Method)*.

2.4.2/ Trial Manufacturing and Material Property Testing of the Binder to be Developed

According to the result of the raw material asphalt selection test, ones with better modifying effect were selected, and the binder for development was manufactured for trial. A general material property test and a rheology characteristics test shown in Table 2.6 were conducted.

*GPC is chromatography by separation using the difference of material permeability into a 3-dimensional mesh structured gel grain.

The obtained value was then compared with the development objective value.

2.5/ Test Results and Considerations

2.5.1/ Raw Material Selection Test

The result of the general material property test is shown in Table 2.7 and its modifying effect evaluation for selection of rubberized raw asphalt is shown in Table 2.8.

The general material property test result of rubberized raw asphalt in Table 2.7 shows that the penetration became lower compared with that of raw asphalt, and its softening point is higher. The degree of these differences depends upon the manufacturer of used raw asphalt or on the penetration grades.

The ductility at low temperature (7°C) has improved greatly by adding rubber. However, the penetration grade 40 to 60 of A in raw asphalt did not show any improvement in its low temperature ductility.

The penetration ratio after evaporation tends to be greater after adding rubber. Four samples out of 14 exceeded the upper limit of 110% set as the standard value in the Asphalt Pavement Manual, and four other samples were very close to the limit (between 105% and 110%). This result is closely related to the raw asphalt. All of the samples of penetration grades 40 to 60 and 60 to 80 of B and C failed; this shows that rubber solubility differs depending on the manufacturers of raw asphalt used.

Toughness tenacity varies drastically with the adding of rubber. The degree of change is quite large with the figure being 1.5 to 2 times that of raw asphalt. The high temperature viscosity became higher by adding rubber. The degree of change was 2 to 3 times the value of raw asphalt.

As far as the modifying effect of rubberized raw asphalt is concerned, Table 2.8 shows clearly that the material properties of rubberized asphalt depend upon the manufacturers of raw asphalt so that, out of the five manufacturers of raw asphalt, the five types were different from the others having penetration grades of 40 to 60 in A, 40 to 60 and 60 to 80 in B, and 40 to 60 and 60 to 80 in C. For example, the penetration ratio of the samples having penetration grade 40 to 60 and 60 to 80 in B and C exceeded 110% after evaporation. Further, the low temperature ductility (7°C) of the sample with 40 to 60 in A remained the same as that of raw asphalt.

2.5.1.1/ Chemical Property Test

1/ Table 2.9 shows the results of the composition analysis. Findings of the test follow.

- **Asphalt manufacturers and asphaltene quantity**

In the 14 samples, the asphaltene quantity of A types having grade 3 penetration is about half that of others and lies between 7.1% and 7.7%.

- **Penetration grade and asphaltene quantity**

There is no relationship between the penetration grade and the asphaltene quantity.

- **Asphalt manufacturers and other components**

The saturated portion differs according to the manufacturer. In particular, A and B have a value about 4% to 10% less than the others. The aromatic portion in A is 9% to 15% higher than others; however, its resin portion is between 19% to 22% regardless of manufacturers or penetration grades.

- **Appearance of separated components**

The sample with penetration grades of 40 to 60 from manufacturer C had peculiarities in the separated component appearance listed as follows.

  - Most of the asphaltene was not in the form of a beautiful brown powder but in black lumps as if carbonated.
  - The saturated portion is usually in the form of a white precipitate like paraffin; however, it was transparent and had fluidity at room temperature (15°C to 20°C).

- The resin portion did not have white suspension at restoration but consisted of an unexpectedly transparent substance.

2/ Molecular weight distribution

Measurement results are shown in Table 2.10. Outstanding points are as follows.

- **Manufacturers and molecular weights**

The molecular weight Table 2.10 suggests a relationship with asphaltene quantity, which is a high-molecular-weight component. Namely, out of five manufacturers' asphalt, the difference between manufacturer A which had a drastically small asphaltene quantity, and other manufacturers is 250 to 400 in molecular weight of Mw, and 60 to 80 in that of Mn. A, having a lesser quantity of asphaltene, has a smaller molecular weight.

- **Manufacturers and molecular weight dispersion rate**

The molecular weight dispersion rate (PdI = Mw/Mn) varies depending on the asphalt manufacturer. It shows a relationship with asphaltene quantity. For example, the dispersion rate of A has a lesser quantity of asphaltene (PdI = 2.26) while B through to E have an average value of 2.57.

2.5.1.2/ Selection of Raw Asphalt for the Binder to be Developed

Based on the test results for the selection of raw asphalt, three types of D having penetration grades 40 to 60, 60 to 80 and 80 to 100, and one type of E with penetration grade 60 to 80 were selected. When selecting the penetration grade, 1) having a small penetration by adding rubber and 2)being abundantly available nation-wide and having a homogenized quality were considered; 60 to 80 was finally selected. At the selection of raw asphalt, two points, such as 1) having a larger toughness tenacity and 2) having a small change in the penetration by the adding of rubber, were considered. Then, for the raw asphalt for the trial manufacture of the binder, the one with the penetration grade 60 to 80 in E was finally selected. Further, in order to see the difference of the modifying effects in raw asphalt, the binders were manufactured on a trial basis after selecting the ones from four manufacturers and after changing types of rubber. The combination of these is shown in Table 2.11.

2.5.2/ Material Property Test for Binders to be Developed

The result of the general material property test for the binders is shown in Table 2.12 and the result of the rheology test in Table 2.13. Those test results may be summarized as follows.

2.5.2.1/ Adding Quantity of Rubber and the Property of Binders to be Developed

Various properties of the binder vary according to the quantity of rubber added. This property has a tendency to be improved as the added quantity increases. However, its viscosity at high temperatures rises and is not favorable in terms of execution.

2.5.2.2/ Moonie Value of Rubber and the Property of the Binder to be Developed

The property of the binder depends greatly on the moonie value of the rubber used as long as the asphalt is the same. As the moonie value becomes larger, its modifying effect in the property becomes greater. However, as the moonie value increases, the mixing or mutual solubility becomes difficult, and it shows a tendency to bubble due to an increased viscosity. Thus, it becomes hard to obtain a homogeneous rubberized asphalt and so test results spread widely. In addition, measurement of the soluble ethane trichloride was not possible for rubber with a moonie value of 150 because it does not easily become a solution and is thick due to the loose gel in rubber. On the contrary, although NR has a high moonie value of 200, it was possible to perform tests because the molecular chains were broken by heat and resolved in the solvent.

2.5.2.3/ Comparison of Property of SBR-Added and NR-Added Binders to be Developed

NR was situated in between SBR's moonie value of 125 and 150. The fraas breaking point which is an indicator of the low temperature property was slightly low. The low temperature ductility (7°C) was smaller than that of SBR. The high temperature viscosity (200°C) was low and about the same as that of SBR 125. This was caused by the fact that NR's molecular structure was disengaged in the heated asphalt. Furthermore, the toughness tenacity was larger compared with that of SBR.

2.5.2.4/ Rheology Characteristics

As for the rheology property of binders (rheology characteristics test), there is some relationship with rubber. Its tendency was to take larger property values as the moonie value increased. For stiffness, the high speed tensile test and the anti-impact property tests data were widely spread. Although there was a slight trend, it was not strong enough to form a relationship with the moonie value.

2.5.2.5/ Evaluation of the Binder to be Developed in the Objective Material Property Value

The material property value of the binder was compared with the objective value. Then, the ones meeting the target value were given o, the ones slightly less than that were given Δ, and the ones far away from that were given x. The result is shown in Table 2.14. According to the result, evaluation of the objective material property value was plotted in terms of the relationship between the moonie values of raw material rubber and adding quantity (Figure 2.3). Furthermore, graphic expressions of evaluation against the development objective value are shown in Figure 2.4 (1) and (2) for each property value.

In SBR- and NR-added binders, the ones capable of meeting the objective material property value or the ones that could be used depending on the use conditions or the execution method (the ones with more than 7 'o' marks) were selected. If they are classified according to the moonie value and rubber-added quantity, the result is like Figure 2.3. Based on these, the range of selection of the binder to be developed was defined. For the purpose of achieving the objective of this research, the following were listed as the ones with great potential for realization:

- adding quantity of 5% to 7% in SBR with moonie value of not greater than 100, and
- adding quantity of 3% to 5% in SBR with a moonie value of not less than 125.

Therefore, those shall have their capability confirmed through mixture tests, serviceability test, and so on after this.

2.6/ Conclusion

2.6.1/ Selection Test of Raw Asphalt

1/ The properties of rubberized asphalt were found to be different in terms of the material property depending on the raw asphalt.

2/ There were some marketed asphalts with a considerably different composition.

2.6.2/ Evaluation of Binders to be Developed Against the Objective Development Value

1/ The one with 7% of SBR added with a moonie value of 70 proved sufficient to make the binder meet all development objectives.

2/ It is necessary to state that there is room for improvement in ones which could meet only one or two development objectives.

POSTSCRIPT

In this report, we have discussed the status quo of modified asphalts in Japan and the development research of rubberized asphalts.

Various modified asphalts in Japan are used mainly as measures for anti-flow, anti-wearing, and so on. Especially, in the application to heavy-traffic roads, any one of them cannot satisfactorily meet requirements. Therefore, the modified asphalts with higher quality are desired.

In addition, for development research on rubberized asphalts we found that predetermined development objective values can be achieved by changing the types of rubber, or adding quantities according to the result of the binder trial manufacture and its quality characteristics test. After this, based on the achievements of the research, the binder to be developed shall be tested by an asphalt mixture test or serviceability test in actual road conditions. Then the quality standard, the mix design standard and the application standard shall be established.

This report was quoted from the research achievements conducted under the name of "Research on Development of Rubberized Asphalt for Anti-Flow and Anti-Wearing Pavement" by joint research of the Public Works Research Institute of the Ministry of Construction, the Foundation of Civil Engineering Research Centre and the Association of Japan Rubberized Asphalt. We are grateful to all the people concerned who gave permission to use this presentation.

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Table 1.1 Demands in Rubberized Asphalt (Converted in Asphalt Unit = Ton)

Type \ Year		1974	1975	1976	1977	1978	1979	1980
Premix	Rubberized asphalt	24,300	22,500	36,900	38,500	41,600	46,500	39,300
	Resin added asphalt	7,500	6,900	6,500	10,100	13,200	13,400	13,700
	Subtotal	31,800	29,400	43,400	48,600	54,800	59,900	53,000
Plant mix		37,500	39,000	50,000	62,500	76,000	85,000	67,500
Total		69,300	68,400	93,400	111,100	130,800	144,900	120,500

Survey by Japan Rubberized Asphalt Institute

Table 1.2 Types of Modified Asphalt

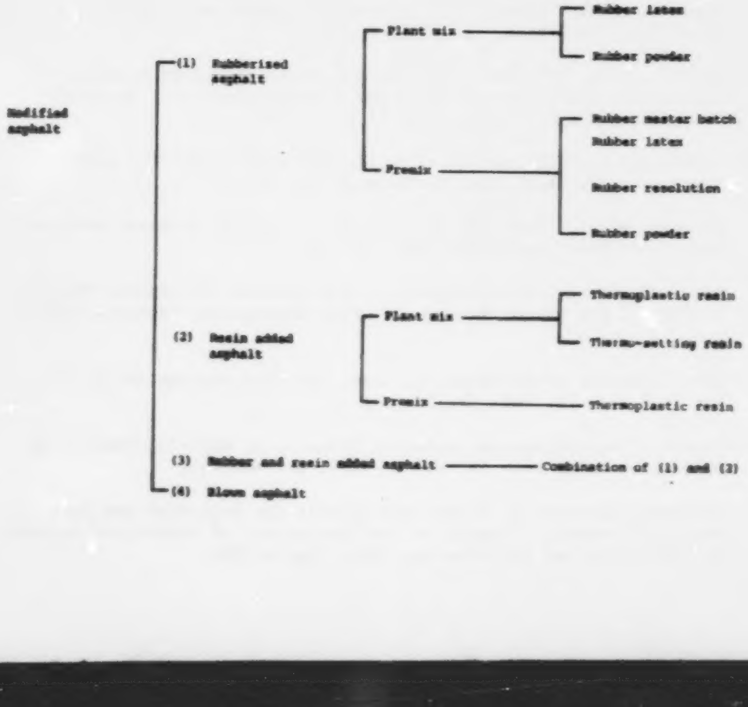

Modified asphalt
(1) Rubberized asphalt
Plant mix
Rubber latex
Rubber powder
Premix
Rubber master batch
Rubber latex
Rubber resolution
Rubber powder
(2) Resin added asphalt
Plant mix
Thermoplastic resin
Thermo-setting resin
Premix
Thermoplastic resin
(3) Rubber and resin added asphalt
Combination of (1) and (2)
(4) Blown asphalt

Table 1.3 Asphalt Modifier

Classi-fication	Type of polymer	Classi-fication	Type of polymer	Classi-fication	Type of polymer
Rubber	Natural rubber (NR)	Thermoplastic elastomer	Stylene butadiene block polymer	Thermoplastic resin	Polyvinyl chloride
	Stylene butadiene copolymer (SBR)		Stylene isoprene block polymer		Polyvinylidene chloride
	Polychloroprene		Polyester elastomer		Polyethylene
	Acrylonitrile butadiene copolymer		Polyurethane elastomer		Polypropylene
	Isobutylene isoprene copolymer		Polyethylene butyl rubber graft polymer		Polystylene
	Polybutadiene		Polyolefin elastomer		Acrylonitrile butadiene stylene copolymer
	Polyisoprene		Ethylene vinyl acetate copolymer		Acrylic resin
	Ethylene propylene 3-d polymer		Ethylene acrylic ester copolymer		Polyamide
	Silicone	Thermo-setting resin	Phenol resin		Polyvinyl acetate
	Fluorine		Urea resin		Polyvinyl alcohol
	Epichlorohydrin polymer		Melamine resin		Saturated polyester
	Polyacrylic ester		Unsaturated polyester resin		Polybutene
	Stylene isoprene copolymer		Polyurethane resin		Petroleum resin
			Epoxide resin		Polycarbonate
			Silicone resin		Fluorine resin
					Natural resin

Table 1.4 Properties of Rubber and Rubberized Asphalt

Item \ Type		Raw asphalt	NR*	SBR	CR	SIR
Rubber quantity (solid portion) %		0	2.0	2.0	2.0	3.0
Penetration (25°C, 100 g, 5 seconds)		85	84	89	96	86
Softening point °C		45.5	50.0	49.5	47.0	46.5
Toughness (25°C) kg-cm		29.3	125.9	36.7	48.8	55.8
Tenacity (25°C) kg-cm		1.1	98.3	8.3	24.4	26.7
Evaporation test	Penetration %	80.8	74.2	73.7	74.5	98.6
	Penetration ratio %	92.4	86.2	92.8	96.1	95.8
Saybolt-Furol viscosity Second	140°C	110	271	215	178	187
	160°C	59	131	110	82	78
	180°C	37	36	57	37	38

Note: NR: Vulcanization when adding

Table 1.5 Rubberized Asphalt Property for Different Raw Asphalt

Item \ Type of raw asphalt	A	B	C	D	E
Penetration (25°C, 100g, 5 seconds)	94	91	89	96	94
Softening point °C	48.0	47.0	48.5	47.5	46.5
Toughness (25°C) kg·cm	81.7	51.5	34.2	22.3	47.8
Tenacity (25°c) kg·cm	53.9	24.4	9.8	5.5	23.1
Ductility (5°C) cm	24	48	60	41	16

Note: Rubber adding quantity: 2% NR (rubber solid portion)

Table 1.6 Marketed Modified Asphalt Property

Item \ Asphalt sample type	Straight asphalt			Semi-blown asphalt		Rubberized asphalt								Resin added asphalt			Rubber and resin added asphalt	
	40∿60	60∿80	80∿100	AC - 80	AC - 140													
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
60°C viscosity (Poise)	5,300	1,950	1,350	7,000	12,230	2,720	3,170	1,950	863	4,810	2,560	1,890	3,080	4,240	4,150	2,640	7,600	6,070
130°C viscosity (Poise)	14.20	8.07	5.57	17.77	20.37	19.67	17.80	13.17	8.63	15.10	16.27	13.87	24.83	19.90	86.33	44.67	36.17	24.63
180°C viscosity (Poise)	0.86	0.67	0.53	0.80	1.10	1.57	1.95	1.31	0.90	1.61	1.79	1.40	2.96	1.06	7.20	4.50	2.99	2.22
Toughness (kg·cm), 25°C	75.9	43.8	31.9	32.6	53.7	141.6	22.3	122.9	10.7	106.8	76.3	35.4	48.5	159.0	60.6	120.0	150.3	68.2
Tenacity (kg·cm), 25°C	8.9	7.4	6.1	4.3	14.3	103.2	0.7	92.2	1.1	46.3	24.4	6.8	11.4	98.0	3.9	35.1	92.2	18.0
Penetration (25°C, 100g, 5 seconds)	43	65	74	57	51	69	77	79	140	50	61	76	64	90	69	80	49	96
Softening point (°C)	53.5	48.5	47.0	54.0	58.5	53.0	52.0	49.5	45.0	51.0	54.0	52.0	57.0	55.0	63.4	61.5	60.0	55.3

Table 2.1 Service Quality Objectives for Binder to be Developed

1) Anti-flow anti-crack property (after passage of 10 mil. cumulative large vehicle traffic volume)

Item	Property	Portion	Current status	Objective standard	Remarks
Anti-flow property	Ruttling	Intersection	20 ①	Less than 10	To limit about 50% of current straight asphalt pavement
		Shortcut	12 ①	Less than 6	
Anti-crack property	Crack ratio (%)	-	17 ②	Less than 9	

Note: ① Road 1981-1

② 29th ministry of construction technical research committee report

2) Anti-wearing property (after passage of 1 mil. cumulative large vehicle traffic volume)

Item	Property	Portion	Current status	Obejctive standard	Remarks
Anti-wearing property	Wearing amount (mm)	-	15 ③	Less than 12	To limit about 80% of current straight asphalt pavement

Note: ③ Road 1980-8 (Service years 43 months. Spike wearing ratio 95%. Average figure in Hokkaido, Feb. 1978)

Table 2.2 Material Property Objectives for Binders to be Developed

Item		New binder (Premix type)		Typical material property of marketed ones (Premix type)		Rubberized asphalt standard in the guideline (Premix type)		Modified asphalt standard for Honshu-Shikoku Bridge (Premix type)	
		Objective material property value	Remarks	Rubberized	Resin added	60~80	80~100	I type	II type
Penetration (25°C, 100g, 5 seconds)		80~100		65	65	60~80	80~100	40~100	40~100
Softening point (R & B) °C		More than 57	(PI + 0.5, - 58.2)	50.5	57.0	48.0~56.0	46.0~56.0	55~65	60~75
Ductility (7°C) cm		More than 30		65	More than 100 (25°C)	More than 30	More than 50	More than 50(15°C)	More than 30 (15°C)
Penetration ratio after evaporation %		Less than 110		102	-	Less than 110	Less than 110	-	-
Thin-film oven test (%)		More than 65	163°C×5 H	75.2	67.2	More than 55	More than 55	More than 65 (163°C, 5 H)	More than 65 (163°C, 5 H)
Fraass breaking point °C		Less than -12		-	-10	-	-	Less than -12	Less than -12
Toughness (25°C) kg-cm		More than 100		91	130	More than 60	More than 80	More than 130	More than 30
Tenacity (25°C) kg-cm		More than 50		56	87	More than 30	More than 25	More than 100	More than 10
Viscosity	60°C Poise	More than 6000		4810	5750			More than 6000	More than 14000
	180°C cst	Less than 300		71	120			Less than 200 (S.F.S.)	Less than 200 (S.F.S.)

Table 2.3 Flow Chart of the Research A

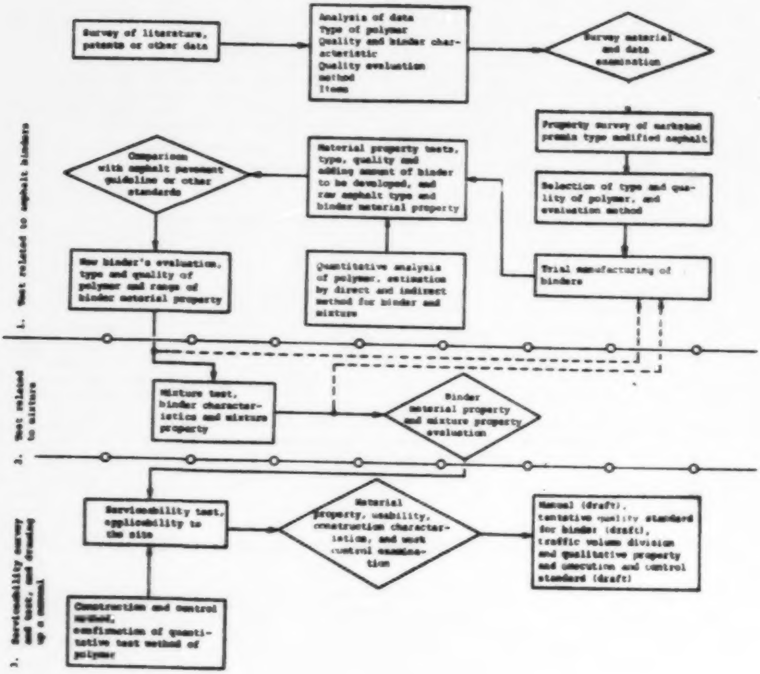

1. Test related to asphalt binders
Survey of literature, patents or other data
Analysis of data
Type of polymer
Quality and binder characteristic
Quality evaluation method
Items
Survey material and data examination
Property survey of marketed premix type modified asphalt
Selection of type and quality of polymer, and evaluation method
Trial manufacturing of binders
Material property tests, type, quality and adding amount of binder to be developed, and raw asphalt type and binder material property
Comparison with asphalt pavement guideline or other standards
New binder's evaluation, type and quality of polymer and range of binder material property
Quantitative analysis of polymer, estimation by direct and indirect method for binder and mixture
2. Test related to mixture
Mixture test, binder characteristics and mixture property
Binder material property and mixture property evaluation
3. Serviceability survey and test, and drawing up a manual
Serviceability test, applicability to the site
Material property, usability, construction characteristics, and work control examination
Manual (draft), tentative quality standard for binder (draft), traffic volume division and qualitative property and execution and control standard (draft)
Construction and control method, confirmation of quantitative test method of polymer

Table 2.4 Basic Material Property of SBR and NR Latex

Type of rubber	Name	Stylene butadiene ratio	Actual moonie value	All solid portion (%)	pH	Surface tension (dyn/cm)	BF viscosity (cp)	Solidified portion (%)	Mechanical stability (%)	Chemical stability ([illegible])
SBR latex	SBR-50	25:75	48.5	49.9	10.4	-	160	0.001	-	-
	SBR-70	25:75	69.0	50.1	10.6	37	137	0.001	0.004	0.23
	SBR-100	25:75	98.0	50.0	10.5	36	160	0.001	0.003	0.24
	SBR-125	25:75	128	50.2	10.5	41	590	0.001	0.001	0.22
	SBR-150	25:75	150	70	10.1	32.8	450	0.003	0.030	0.10
NR latex	NR-200	Measured actual moonie value 200								

Note: Moonie value: viscosity value of unvulcanised rubber

Table 2.5a Trial Manufacturing Conditions for the Binder to be Developed

Process	Condition
Asphalt temperature	165 ± 5°C
Latex adding time	2 ~ 3% : Approx. 20 min. 5 ~ 7% : Approx. 40 min.
Agitation time	1.5 ~ 2.0 h
Agitation speed	350 r.p.m.
Location of fin	5 cm below asphalt surface
Sample weight	10 kg
Rubber adding	Added in form of latex and converted into solid weight (%)

Table 2.5b Combination and Substance of Raw Material Asphalt Test

Test purpose	Asphalt		Latex			Total	Test substance
	Penetration	Manufacturer	Type	Mooney viscosity	Adding quantity		
Raw asphalt selection test	40/60 60/80 80/100	A B C D E	SBR	50	0 3	28	° General material property test ° Chemical property test (composition and molecular weight distribution)

Note: 1 40/60: 4 manufacturers only

2 Adding amount: converted solid rubber portion distributed weight (%)

Table 2.6 Test Method and Conditions

Test division	Test items	Outline	Test method and conditions
General material property test	Penetration, softening point, ductility, flash point, toughness, and tenacity	Test general material property of asphalt	JIS and other test methods related to asphalt
Properties such as rheology characteristics test	Recovery characteristics	Examine recovery characteristics of binder	(Method) Using instron and obtain shrinkage rate and time at the free and after stretched (Conditions) Measuring temperature: 15°C Ductility: 200% Stretching speed: 500 mm/min.
	Simple viscosity test	Check viscosity characteristics from temperature and time	(Method) Using Foratball testing equipment and obtain consistency around the softening point (Conditions) Measuring temperature: 60°C Sample quantity: 100 g
	Anti-impact test	Check anti-impact property at low temperature	(Method) Using Beige impact testing equipment and obtain value for binders (Conditions) Judging temperature: -5, 0, +5°C Sample: φ20 mm × H30 mm
	Stress adaptation test	Show stress adaptability in and check its characteristics	(Method) Using instron and obtain stress adaptation at each temperature (Conditions) Judging temperature: 10, 15, 20°C Stretching speed: 500 mm/min. Strain deformation: 10 mm Sample: Using ductility form
	Stiffness characteristics	Check rheology characteristics at low temperature	(Method) Using sliding plate rheometer and obtain stiffness at low temperature area (Conditions) Stiffness: Temperature: 0, 20°C Sample thickness: 10 mm Weight: 50∿2000 g
	High speed tension test	Show tension in terms of temperature and check its characteristics	(Method) Using instron and conduct high speed stretching test, then obtain spreading strength of binder (Conditions) Measuring temperature: 10, 15, 20°C Stretching speed: 500 mm/min. Sample: Using ductility form

Table 2.7 Raw Asphalt Selection, General Material Property Test Result

Manufacturer	Sign	Grade	Penetration		Softening point		Ductility (7°C)		Ductility (15°C)		Penetration ratio after evaporation (163°C 5H)		Thin-film oven weight change (163°C 5H)%		Thin-film oven weight change (180°C 2.5H)%		After thin-film oven (163°C 5 H)		Penetration after thin-film oven (180°C 2.5 H)		Penetration change rate after thin-film oven (180°C 2.5 H)	
			Base	3% SBR added	Base	3% SBR added	Base	3% SBR added	Base		Base	3% SBR added	Base	3% SBR added	Base	3% SBR added	Penetration Base	Softening point Base	Base	3% SBR added	Base	3% SBR added
A	a	40/60	57	47	49.0	51.5	0	0	100+		110	98	+0.11		-0.08	+0.05	38	52.0	40	44	70.2	87.2
	b	60/80	64	64	46.0	48.5	0	11	100+		92	102	+0.11		+0.07	+0.05	47	49.5	51	58	79.6	90.6
	c	80/100	90	87	44.0	47.0	11	100+	100+		76	79	+0.12		+0.09	+0.05	69	47.0	71	78	79.5	88.3
B	a	40/60	46	46	52.0	56.5	0	10	29.0		93	138	+0.11		+0.06	+0.06	33	56.0	38	38	82.6	82.6
	b	60/80	67	60	50.0	56.0	6	100+	100+		92	116	+0.05		+0.02	00.00	46	53.5	46	50	68.7	85.3
	c	80/100	91	79	45.0	47.0	7	100+	100+		93	86	+0.01		-0.01	-0.005	68	49.5	62	64	69.1	81.0
C	a	40/60	53	51	50.0	55.0	1	100+	100+		96	106	-0.02		-0.09	-0.08	37	55.0	39	43	73.6	83.3
	b	60/80	66	61	48.5	53.5	5	100+	100+		93	112	+0.03		-0.02	-0.06	43	53.1	46	49	68.7	80.3
	c	80/100	82	77	46.0	50.5	6	100+	100+		96	103	+0.04		+0.01	-0.03	55	50.5	56	60	68.3	77.9
D	a	40/60	50	51	49.5	54.5	0	12	100+		105	100	+0.10		+0.07	+0.04	36	53.5	39	42	78.0	82.4
	b	60/80	72	65	47.0	52.0	6	100+	100+		78	106	+0.10		+0.07	+0.04	60	51.0	51	55	70.6	84.6
	c	80/100	89	78	45.0	50.5	7	100+	100+		97	103	+0.11		+0.07	+0.06	57	49.5	63	65	70.8	83.3
E	b	60/80	67	63	47.5	52.0	6	100+	100+		95	100	+0.10		+0.00	+0.03	47	52.0	50	51	74.6	81.0
	c	80/100	90	83	44.5	49.5	10	100+	100+		93	170	+0.11		+0.07	-0.02	62	49.0	63	68	70.0	83.1

Manufacturer	Sign	Grade	Softening point after thin-film oven (180°C 2.5H)		Flash point		Specific gravity		Tri-chloride ethane soluble portion %		Fraas breaking point		Ash content %		Toughness kg·cm		Tenacity kg·cm		60°C viscosity Poise		180°C viscosity cst	
			Base	3% SBR added	Base	3% SBR added	Base	3% SBR added	Base	3% SBR added	Base	3% SBR added	Base	3% SBR added	Base	3% SBR added	Base	3% SBR added	Base	3% SBR added	Base	3% SBR added
A	a	40/60	51.0	52.0	352	340	1.0431	1.0300	99.95	99.49	- 3	- 3	0.03	0.21	96.3	125.6	21.5	44.3	2562	4012	38	112
	b	60/80	48.5	50.5	345	346	1.0402	1.0363	99.97	99.91	- 3	- 7	0.08	0.16	55.6	75.9	16.9	28.6	1500	2656	47	87
	c	80/100	46.0	47.5	351	342	1.0362	1.0313	99.93	99.89	- 9	-10	0.10	0.18	31.9	47.5	10.5	20.5	1033	1839	32	106
B	a	40/60	55.0	58.0	349	343	1.0366	1.0328	99.92	99.8	- 9	- 8	0.17	0.36	91.8	92.1	15.9	27.0	4660	10821	85	190
	b	60/80	53.0	55.0	327	330	1.0403	1.0371	99.91	99.80	-11	-10	0.10	0.18	58.4	70.7	12.7	21.8	2062	7769	58	140
	c	80/100	49.0	50.0	331	336	1.0294	1.0265	99.96	99.85	-13	- 9	0.13	0.19	33.8	50.0	8.3	21.1	1031	2660	40	127
C	a	40/60	58.0	58.5	362	301	1.0370	1.0254	99.96	99.96	- 7	- 5	0.30	0.32	74.6	105.4	12.7	31.3	4331	11289	56	138
	b	60/80	52.5	55.0	339	335	1.0301	1.0287	99.91	99.83	- 9	-10	0.17	0.23	54.3	71.0	10.3	22.7	2604	6353	42	109
	c	80/100	50.0	51.5	331	325	1.0297	1.0266	99.71	99.17	-11	- 9	0.13	0.25	37.6	60.3	6.3	21.2	1080	4381	42	104
D	a	40/60	53.0	55.0	347	339	1.0363	1.0334	99.71	99.89	- 2	- 9	0.04	0.31	98.4	105.7	21.3	33.1	3383	6870	53	125
	b	60/80	50.5	52.5	354	341	1.0238	1.0270	99.90	99.06	- 8	-11	0.09	0.15	56.7	71.2	14.4	21.8	2128	4536	50	114
	c	80/100	48.0	51.5	337	335	1.0236	1.0065	99.71	99.86	-10	-10	0.09	0.27	36.4	56.3	10.3	20.7	1619	3317	39	106
E	b	60/80	51.5	54.0	345	338	1.0306	1.0294	99.89	99.86	- 9	- 9	0.08	0.22	50.1	74.0	16.6	36.1	2800	5464	57	136
	c	80/100	48.0	50.0	340	336	1.0289	1.0239	99.76	99.88	-12	-10	0.15	0.27	36.1	52.0	11.9	30.8	1881	3676	39	129

Note: SBR adding amount: converted solid rubber distributed weight

Table 2.8 Evaluation of Modifying Effect in Selection of Raw Asphalt and General Material Property Test

Manu-facturer	Sign	Grade	Penetration	Softening point	Ductility (7°C)	Penetration ratio after evaporation (163°C)	Thin-film oven weight change (163°C, 2.5h)	Penetration after thin-film oven (163°C, 2.5h)	Penetration change rate after thin-film oven (163°C, 2.5h)	Softening point after thin-film oven (163°C, 2.5h)	Flash point
A	a	40/60	△	○	×	○	○	○	○	○	○
	b	60/80	○	○	△	○	○	○	○	○	○
	c	80/100	○	○	○	○	○	○	○	○	○
B	a	40/60	○	○	△	×	○	○	○	○	○
	b	60/80	○	△	○	×	○	○	○	○	○
	c	80/100	△	○	○	○	○	○	○	○	○
C	a	40/60	○	○	○	×	○	○	○	△	○
	b	60/80	○	○	○	×	○	○	○	○	○
	c	80/100	○	○	○	○	○	○	○	○	○
D	a	40/60	○	○	△	○	○	○	○	○	○
	b	60/80	○	○	○	○	○	○	○	○	○
	c	80/100	△	△	○	○	○	○	○	○	○
E	b	60/80	○	○	○	○	○	○	○	○	○
	c	80/100	△	○	○	○	○	○	○	○	○

Manu-facturer	Sign	Grade	Specific gravity	Tri-chloride ethane soluble portion	Fraas breaking point	Ash content	Toughness	Tenacity	60°C viscosity	180°C viscosity
A	a	40/60	○	○	×	○	○	○	○	○
	b	60/80	○	○	○	○	○	○	○	○
	c	80/100	○	○	○	○	△	△	○	○
B	a	40/60	○	○	○	○	○	○	○	○
	b	60/80	○	○	○	○	○	△	○	○
	c	80/100	○	○	○	○	○	△	○	○
C	a	40/60	○	○	×	○	○	○	○	○
	b	60/80	○	○	○	○	○	△	○	○
	c	80/100	○	○	○	○	○	△	○	○
D	a	40/60	○	○	○	○	○	○	○	○
	b	60/80	○	○	○	○	○	△	○	○
	c	80/100	○	○	○	○	○	△	○	○
E	b	60/80	○	○	○	○	○	○	○	○
	c	80/100	○	○	○	○	○	△	○	○

Test item	Evaluating criteria			Test item	Evaluating criteria		
	○	△	×		○	△	×
Penetration	Change rate less than ±10%	Change rate more than ±11%		Specific gravity	More than 1.000		
Softening point	·	·		Tri-chloride ethane soluble portion	More than 99%		
Ductility (7°C)	More than 100 cm	99~10 cm	Less than 9 cm	Fraas breaking point	Less than -7°C		More than -6°C
Penetration ratio after evaporation	Less than 110%		More than 111%	Ash content	Less than 1.0%		
Thin-film oven weight change (163°C, 2.5 H)	Less than 0.6%			Toughness	More than 50 kg-cm	Less than 49 kg-cm	
Penetration after thin-film oven (163°C, 2.5 H)	Change rate more than 50%			Tenacity	More than 25 kg-cm	Less than 24 kg-cm	
Penetration change rate after thin-film oven (163°C, 2.5 H)	More than 60%			60°C viscosity	Whichever goes higher		
Softening point after thin-film oven (163°C, 2.5 H)	Change rate less than 9%	Change rate more than 10%		180°C viscosity	Less than 200 cSt		
Flash point	More than 260°C						

Table 2.9 Result of Raw Asphalt Composition Analysis

Penetration	40~60				60~80				80~100			
Components / Manufacturer	Asphalt	Saturated	Aromatic	Resin	Asphalt	Saturated	Aromatic	Resin	Asphalt	Saturated	Aromatic	Resin
A	7.1	12.3	8.1	22.5	7.7	14.8	55.9	21.6	7.2	15.6	55.6	21.6
B	15.8	16.2	47.2	20.8	17.1	15.1	48.1	19.7	11.5	17.1	49.2	22.2
C	14.6	21.9	41.1	22.4	16.0	18.7	43.8	21.5	14.8	20.0	45.1	20.1
D	14.4	17.7	46.3	21.6	13.8	19.5	45.4	21.3	14.7	22.7	43.6	19.0
E					14.8	18.2	47.3	19.7	12.0	19.5	47.8	20.7

Table 2.10 Result of Raw Asphalt Molecular Weight Measurement

Sample name	Average molecular weight Mw	Molecular weight Mn	Molecular weight distribution PDI (Mw/Mn)	Asphaltene amount (%)
A-a (40 ~ 60)	1040	466	2.23	7.1
A-b (60 ~ 80)	1081	474	2.28	7.7
A-c (80 ~ 100)	1085	480	2.26	7.2
B-a (40 ~ 60)	1564	606	2.58	15.8
B-b (60 ~ 80)	1582	584	2.71	17.1
B-c (80 ~ 100)	1309	476	2.75	11.5
C-a (40 ~ 60)	1335	480	2.78	14.6
C-b (60 ~ 80)	1461	551	2.65	16.0
C-c (80 ~ 100)	1388	518	2.68	14.8
D-a (40 ~ 60)	1290	558	2.31	14.4
D-b (60 ~ 80)	1326	554	2.39	13.8
D-c (80 ~ 100)	1311	515	2.54	14.7
E-b (60 ~ 80)	1317	540	2.44	14.8
E-c (80 ~ 100)	1287	534	2.41	12.0

Table 2.11 Combination and Substance for the Test of the Binder to be Developed

Testing purpose	Asphalt		Latex			Total		Test substance
	Penetration	Manufacturer	Type	Mooney viscosity	Adding amount			
Material property test for the binder to be developed	60/80	E	SBR	50 75 100 125 150	2 3 5 7	20	24	• General material property test • Rheology test characteristics
		A B C D	SBR	150	3	4		
	40/60	E	NR	200	2 3 5	3	7	
		A B C D	NR	200	3	4		

Note: Adding amount: converted solid rubber weight

Table 2.12 Result of General Material Property Test for the Binder to be Developed

[illegible]	Type of [illegible]	[illegible]	Adding amount	Penetration	Softening point °C	Ductility cm (7°C)	Penetration ratio after [illegible] % (163°C)	Thin-film oven weight change % (163°C, 5 h)	Penetration of after thin-film oven (163°C, 5 h)	Penetration change ratio after thin-film oven % (163°C 5 h)	Softening point of after thin-film oven (163°C, 5 h)	Flash point °C (Cleveland open method)	Specific gravity	Tri-chloride ethane soluble portion %	Frass breaking point °C	Ash content %	Toughness kg-cm	Tenacity kg-cm	Viscosity 60°C Poise	Viscosity 180°C cSt
		60	0	67	47.5	6	95	+0.08	60	90.7	51.5	341	1.0305	99.96	- 9	0.08	26.1	14.6	2.080	57
			2	63	49.5	100+	107	+0.07	50	79.4	53	342	1.0302	99.87	- 9	0.12	47.4	19.0	4.080	91
			3	63	52	100+	108	+0.06	51	81.0	56	343	1.0290	99.86	-11	0.24	73.6	21.7	5.083	136
			5	59	57	100+	137	+0.03	55	93.2	55.5	328	1.0280	99.83	-13	0.06	76.3	30.9	9.676	211
			7	57	61.5	100+	166	+0.05	50	87.7	60	337	1.0277	99.76	-11	0.43	80.0	35.2	17.763	336
		80	2	66	50.5	63	100	+0.06	60	83.3	51.5	346	1.0302	99.80	-10	0.06	66.4	22.3	5.148	95
			3	58	53	100+	135	+0.06	51	87.9	51.5	350	1.0280	99.86	-10	0.37	68.3	33.6	5.885	131
			5	55	56	100+	167	+0.03	51	92.7	56	336	1.0284	99.62	-10	0.31	95.5	46.7	11.930	206
	B		7	55	63	100+	162	-0.01	60	87.3	63.5	346	1.0280	99.37	-12	0.43	109.7	54.3	21.366	280
E	B	100	2	63	53	14	126	+0.06	53	85.5	51.5	340	1.0300	99.91	-11	0.25	90.0	43.5	4.706	160
			3	61	54	100+	111	+0.06	52	85.2	53	345	1.0271	99.74	- 9	0.08	96.3	52.5	6.047	115
	B		5	55	57	100+	165	+0.03	51	92.7	57	341	1.0270	99.71	-13	0.00	111.3	71.5	13.923	296
			7	53	66.5	100+	127	+0.01	47	88.7	64.5	340	1.0264	99.60	-13	0.30	130.0	77.0	19.064	477
		120	2	62	51.5	30	100	+0.05	56	87.1	51	342	1.0290	99.85	-10	0.23	85.1	40.0	5.230	73
			3	61	56	100+	135	+0.06	56	88.5	52.5	343	1.0279	99.72	-11	0.17	101.3	55.3	6.960	167
			5	53	61.5	100+	136	+0.07	53	100.0	58.5	330	1.0260	99.60	- 9	0.29	122.5	71.2	10.006	301
			7	52	68.5	100+	98	-0.02	51	98.1	68.5	336	1.0271	99.20	-16	0.13	125.4	79.7	23.087	766
		130	2	60	56.5	22	129	+0.05	62	103.1	52.5	337	1.0275	-	- 8	0.11	162.5	103.2	9.232	912
			3	50	57.5	21	107	+0.05	52	89.7	54	340	1.0282	-	-11	0.14	153.4	96.2	9.982	1318
			5	56	63	50	118	+0.06	47	87.0	63	347	1.0255	-	-12	0.61	200.6	200.7	20.762	-
			7	57	74.5	100+	115	+0.03	56	98.2	71.5	331	1.0081	-	-16	0.22	339.5	277.4	60.776	-
A		150	3	60	55.5	12	100	+0.07	56	76.3	51.5	300	1.0363	-	- 7	0.11	132.0	101.1	5.086	327
B				56	58.5	26	130	+0.02	52	95.3	56.5	305	1.0300	-	- 9	0.08	261.6	99.6	22.327	263
C				61	58.5	22	112	-0.01	51	83.6	55	305	1.0290	-	- 7	0.18	155.1	98.0	10.080	1296
D				66	57.5	27	104	+0.05	58	67.9	52.5	294	1.0235	-	-10	0.16	166.2	111.0	8.404	935
E		200	2	61	52.5	10	88	+0.06	56	91.0	49.5	326	1.0310	99.83	-11	0.33	127.0	66.3	4.148	123
			3	66	54.5	17	122	+0.06	57	95.0	50	346	1.0296	99.85	-14	0.37	137.4	80.0	7.815	160
	B		5	58	58	14	110	+0.03	58	105.4	51	383	1.0279	99.73	-16	0.35	163.0	88.6	5.277	138
A		200	3	77	68	24	109	+0.06	66	86.7	68.5	369	1.0366	99.80	- 7	0.15	126.0	98.7	3.490	159
B	B			99	70.5	20	109	+0.05	77	96.6	70.5	360	1.0300	98.49	- 9	0.28	130.7	76.1	5.007	161
C				56	57.5	10	111	+0.00	56	100.0	50.5	327	1.0701	90.41	- 8	0.11	160.0	67.0	10.034	179
D				60	54.5	15	111	+0.03	57	95.0	60.5	327	1.0230	99.94	-13	0.24	177.2	125.3	3.260	99

Note: 1 : Adding amount; converted solid rubber weight (%).
2 : [illegible]

Table 2.13 Rheology Characteristics Test

Asphalt	Type of rubber	Sample	Adding weight	Recovery characteristic 15°C [illegible] recovery rate after 10 seconds (%)	[illegible] viscosity test 60°C [illegible]	Anti-impact test Temp. [illegible] (times)			Flexure [illegible] test [illegible] (seconds)			Stiffness Sliding plate test Loading time: 10 seconds (kg/cm²)				60°C viscosity Poise 60°C	High speed tension test maximum strength (kg)		
						-5°C	0°C	+5°C	10°C	15°C	20°C	-10°C	0°C	10°C	20°C		10°C	15°C	20°C
B	[illegible]	50	0	5.7	27.25	3	3	6	-	3.00	2.10	200	49.3	1.67	2.457×10^{-1}	3.900	(1.8)	6.53	3.80
			1	10.0	42.05		4			4.00	3.00		65.0		3.095×10^{-1}	4.000		9.03	
			3	14.5	84.05	3	4	5	0.20	3.44	3.40		13.7		1.070	5.983	(17.1)	8.33	3.01
			5	20.3	111.70		9			5.90			31.8		3.633×10^{-1}	9.474		14.37	
			7	26.3	300.21		20			5.00			28.4		1.733×10^{-1}	17.703		8.30	
		70	2	11.0	49.56		3			2.44			38.4		3.330×10^{-1}	5.110		7.17	
			3	20.6	66.35	3	4		3.44	3.00	3.90		90.1		2.483×10^{-1}	3.850	(10.7)	9.13	6.31
			5	25.9	118.75		9			5.26			40.8		1.532×10^{-1}	11.930		10.47	
			7	26.7	130.77		9			6.07			10.5		4.776×10^{-1}	21.301		11.05	
		100	2	12.5	64.00		5			3.05			34.5		2.000×10^{-1}	4.766		5.47	
			3	15.1	68.40	2	4	6	3.30	3.90	3.12		40.0		2.123×10^{-1}	6.007	16.9	8.03	3.73
			5	20.0	170.61		3			4.92			30.0		3.375×10^{-1}	13.933		11.00	
			7	25.4	202.13		15			6.00			27.2		0.671	29.004		11.37	
		125	2	18.6	63.00		5			3.00			30.2		1.747	5.239		8.07	
			3	20.3	63.07	5	4	15	6.06	6.36	3.20		60.0		3.083×10^{-1}	6.000	21.0	9.67	3.40
			5	26.3	100.07		6			5.34			80.1		6.637×10^{-1}	15.006		10.67	
			7	30.9	256.73		11			7.30			60.0		9.160×10^{-1}	23.037		11.30	
		150	2	30.2	75.06		6			6.04			27.2		3.900×10^{-1}	9.233		7.60	
			3	30.4	105.00	3	6	7	0.42	3.30	3.05	205	95.5	1.00	2.706×10^{-1}	9.056	21.4	7.17	3.03
			5	95.1	670.00		10			6.04			25.0		1.000	20.703		8.53	
			7	26.8	780.27		14			6.30			20.7		6.061	60.776		11.17	
A		150	3	15.3	55.05		4			2.04			97.6		2.120	5.005		9.53	
B				20.5	165.36		4			6.04			47.5		3.207×10^{-1}	13.317		8.17	
C				27.1	63.35		6			4.80			27.0		4.090×10^{-1}	10.750		7.90	
D				19.6	360.00		6			3.00			65.4		2.073×10^{-1}	8.004		7.10	
B	[illegible]	200	2	16.0	40.30		5			3.22			97.6		3.030×10^{-1}	4.142		7.07	
			3	20.3	40.15	4	5	8	6.26	6.00	2.03		30.2		3.132×10^{-1}	7.065	19.0	9.13	6.39
			5	23.6	70.90		8			6.06			30.3		3.631×10^{-1}	12.109		8.03	
A	[illegible]	200	5	5.6	60.70		4			1.04			65.7		5.106×10^{-2}	3.090		7.80	
B				14.8	100.76		6			5.92			30.0		3.031	5.007		7.07	
C				23.6	60.07		6			6.04			23.0		4.080	10.934		8.70	
D				20.[illegible]	77.31		6			0.33			90.0		1.075×10^{-1}	3.200		7.60	

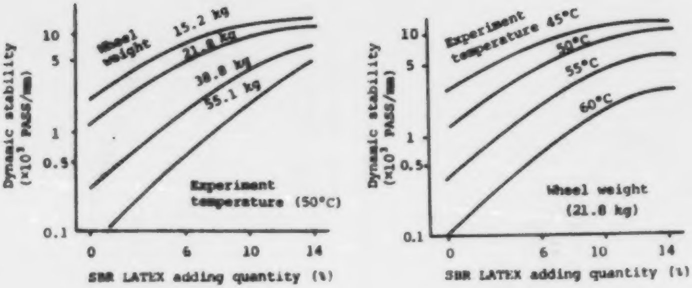
Note: 1. Adding amount: converted solid rubber weight (%)

2. -: unmeasurable

Table 2.14 Evaluation of the Binder to be Developed against the Development Objective

Asphalt	Type of rubber	Penetration value	Adding amount	Penetration 25°C (50 ~ 100)		Softening point (More than 57)		Ductility 7°C (More than 90)		Penetration ratio after evaporation (Less than 110)		Penetration change rate after thin-film oven (More than 65)		Fraass breaking point (Less than -12)		Toughness (More than 100)		Tenacity (More than 50)		Viscosity 60°C Poise (More than 6000)		Viscosity 180°C cst (Less than 300)	
				Evaluation	Measured value	Evaluation	Measured value	Evaluation	Measured value	Evaluation	Measured value	Evaluation	Measured value	Evaluation	Measured value	Evaluation	Measured value	Evaluation	Measured value	Evaluation	Measured value	Evaluation	Measured value
[illegible]	[illegible]	50	0	○	67	×	47.5	×	6	[illegible]	95	×	90.7	×	- 9	×	56	×	14	×	2560	○	57
			2	○	63	×	48.5	○	100+	○	107	○	79.4	×	- 9	×	67	×	19	×	4700	○	91
			3	○	63	×	52	○	100+	○	108	○	81.0	Δ	-11	×	73	×	25	○	6000	○	136
			5	○	59	○	57	○	100+	×	137	○	93.2	○	-13	×	76	×	30	○	9560	○	211
			7	○	57	○	61.5	○	100+	○	106	○	87.7	Δ	-11	×	82	×	38	○	17000	○	234
		70	2	○	60	×	50.5	○	100+	○	109	○	83.3	Δ	-10	×	66	×	22	×	5100	○	85
			3	○	59	×	53	○	51	×	125	○	87.5	Δ	-10	×	82	×	33	Δ	5900	○	131
			5	○	58	○	58	○	100+	×	167	○	92.7	Δ	-10	Δ	95	Δ	46	○	11900	○	[illegible]
			7	○	58	○	63	○	100+	○	103	○	87.3	○	-12	○	105	○	54	○	23400	○	300
		100	2	○	62	×	53	×	14	×	126	○	86.5	Δ	-11	Δ	98	×	63	×	4700	○	108
			3	○	61	×	54	○	100+	Δ	111	○	85.2	×	- 9	Δ	96	○	53	○	6000	○	115
			5	○	55	○	57	○	100+	×	145	Δ	61.0	○	-13	○	111	○	71	○	13000	○	296
			7	○	53	○	60.5	○	100+	×	127	○	88.7	○	-13	○	130	○	77	○	19700	×	477
		125	2	○	62	×	51.5	×	20	○	109	○	93.1	Δ	-10	×	85	×	48	×	5200	○	73
			3	○	61	×	54	○	100+	×	135	○	88.5	Δ	-11	○	101	○	56	○	8000	○	167
			5	○	53	○	61.5	○	100+	×	136	○	100.0	×	- 9	○	122	○	71	○	16000	Δ	301
			7	○	53	○	68.5	○	61	○	98	○	88.1	○	-14	○	135	○	79	○	13000	×	786
		150	2	○	60	○	58	×	22	×	129	○	83.3	×	- 8	○	162	○	103	○	9000	×	912
			3	○	58	○	57.5	×	21	○	107	○	89.7	Δ	-11	○	153	○	90	○	16000	×	1318
			5	○	54	○	63	○	90	Δ	110	○	87.0	○	-12	○	290	○	240	○	20000	×	-
			7	○	57	○	70.5	○	100+	Δ	115	○	90.2	Δ	-10	○	339	○	277	○	60000	×	-
A		150	3	○	60	×	55.5	×	12	○	100	○	78.3	×	- 7	○	152	○	101	×	5000	×	317
B				○	54	○	59.5	×	26	×	130	○	96.3	×	- 9	○	161	○	99	○	13300	×	963
C				○	61	○	58.5	×	22	Δ	112	○	83.6	×	- 7	○	155	○	96	○	10000	×	1336
D				○	66	○	57.5	×	27	○	106	○	87.9	Δ	-10	○	165	○	111	○	6500	×	930
E	[illegible]	200	2	○	61	×	52.5	×	10	○	98	○	91.0	Δ	-11	○	127	○	84	×	6200	○	123
			3	○	60	×	54.5	×	17	×	122	○	93.0	○	-16	○	137	○	90	○	7000	○	165
			5	○	56	○	58	×	14	○	110	○	88.4	○	-16	○	163	○	88	○	13140	×	338
A			3	○	77	×	49	×	20	○	100	○	85.7	×	- 7	○	126	○	90	×	7800	○	190
B				○	58	○	59.5	×	10	×	138	○	95.6	×	- 9	○	130	○	76	×	5000	○	180
C				○	56	○	57.5	×	10	Δ	111	○	100.0	○	- 8	○	146	○	87	○	10900	○	170
D				○	60	×	54.5	×	15	Δ	111	○	95.0	○	-13	○	177	○	125	×	1200	○	90
Modified asphalt	[illegible] asphalt C		F	○	73	×	50	○	80	Δ	111	○	81.6	○	-12	×	80	Δ	47	×	4000	○	185
			G	○	70	×	48.5	○	100+	Δ	103	○	84.3	Δ	-10	×	76	×	25	×	1600	○	130
	Resin rubber added asphalt b		G	×	49	○	58	×	12	Δ	115	○	86.5	Δ	-11	○	111	○	50	○	6600	○	177
			H	○	58	○	57.5	×	26	○	100	○	86.2	○	-14	○	176	○	121	○	9400	○	178
	Resin added asphalt a		F	○	54	○	57.5	×	16	○	102	○	81.5	Δ	-11	○	129	○	71	○	6400	○	141
			I	○	55	○	62.5	×	21	×	127	○	90.9	Δ	-11	○	201	○	134	○	12700	○	177
Evaluation standard		○		50 ~ 100		More than 57		More than 90		Less than 110		More than 65		Less than -12		More than 100		More than 50		More than 6000		Less than 300	
		Δ		47 ~ 49 101 ~ 106		More than 54 Less than 57		More than 45 Less than 90		More than 111 Less than 121		More than 61 Less than 65		-10 ~ -11		More than 90 Less than 100		More than 45 Less than 50		Less than 6000 More than 5700		301 ~ 315	
		×		Less than 46 More than 107		Less than 53		Less than 44		More than 122		Less than 61		More than -9		Less than 89		Less than 45		Less than 5700		More than 316	

Note: Adding amount; converted solid rubber weight (%)



(a) Wheel weight effect on dynamic stability

(b) Temperature effect on dynamic stability

Fig. 1.1 SBR Effects on Wheel-Tracking Test Result

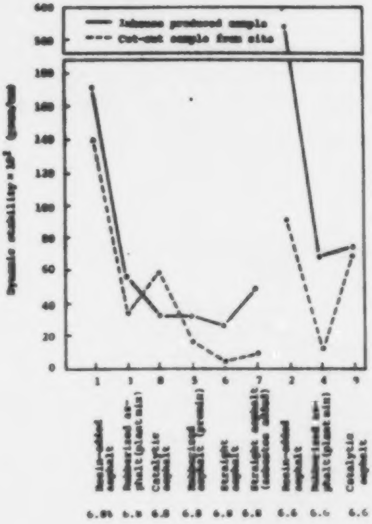


Fig. 1.2 Wheel-Tracking Test Results of Various Asphalts

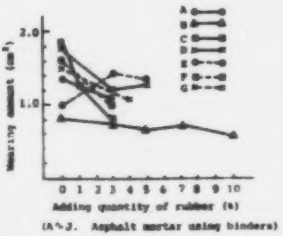


Fig. 1.3 Effect of Rubber on Ravelling

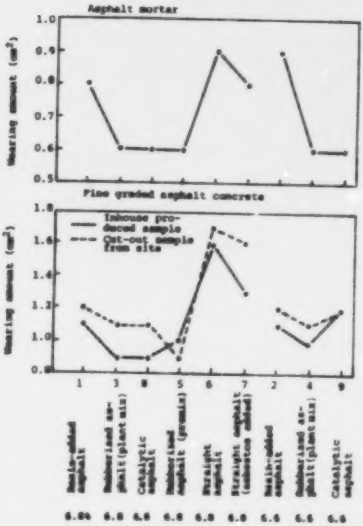
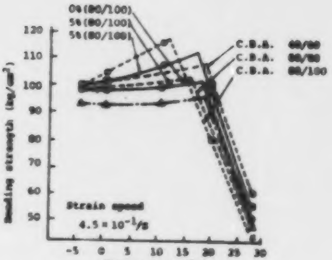
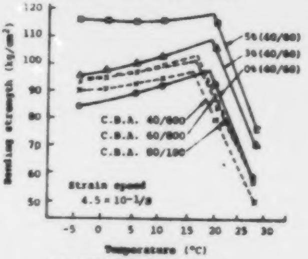


Fig. 1.4 Ravelling Test Result of Various Modified Asphalt



(a) Asphalt concrete mixture



(b) Rolled asphalt mixture

Fig. 1.5 SBR Effect on Bending Strength

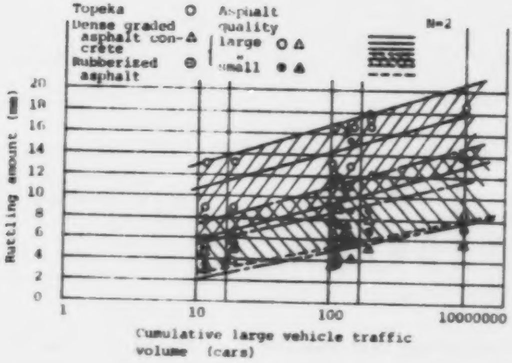


Fig. 1.6 Result of Okabe Experimental Pavement

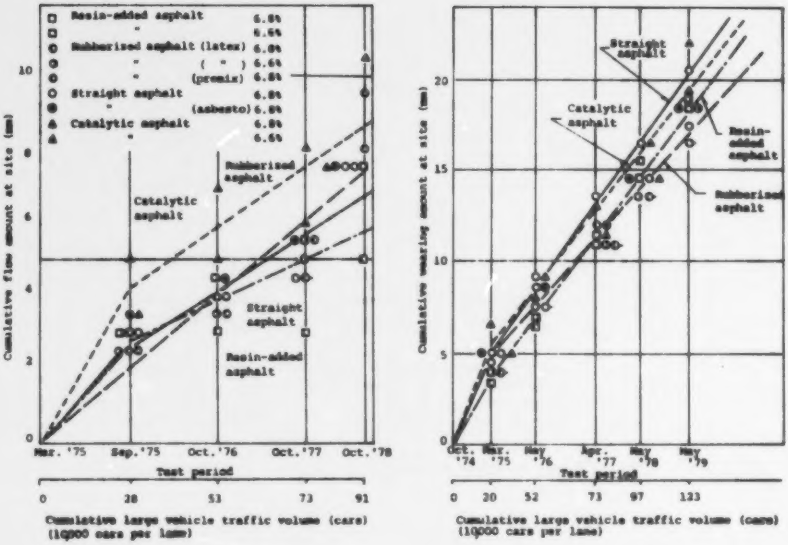


Fig. 1.7 Result of Kohjunai Experimental Pavement

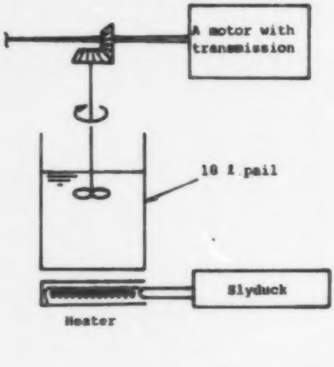

A motor with transmission
10 ℓ pail
Slyduck
Heater

Fig. 2.1 Indoor Production Equipment

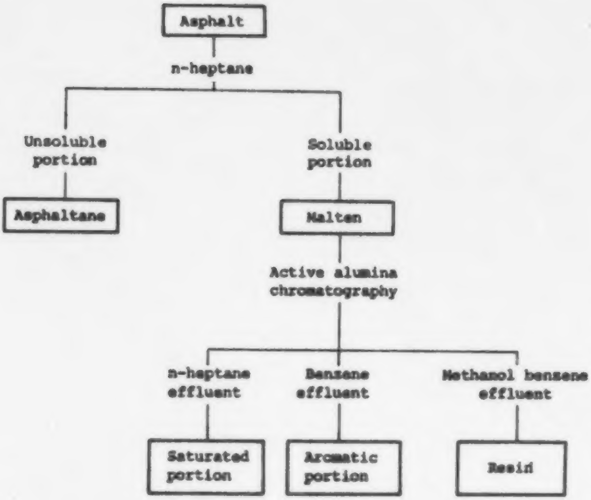

Asphalt
n-heptane
Unsoluble portion
Soluble portion
Asphaltane
Malten
Active alumina chromatography
n-heptane effluent
Benzene effluent
Methanol benzene effluent
Saturated portion
Aromatic portion
Resin

Fig. 2.2 Composition Analysis Method of Asphalt (Iijima Method)

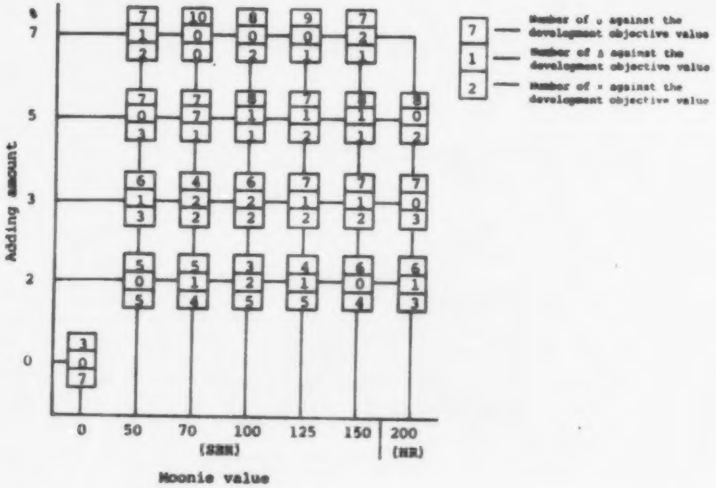


Fig. 2.3 Evaluation of Objective Material Property Value at Moonie Value and Adding Amount

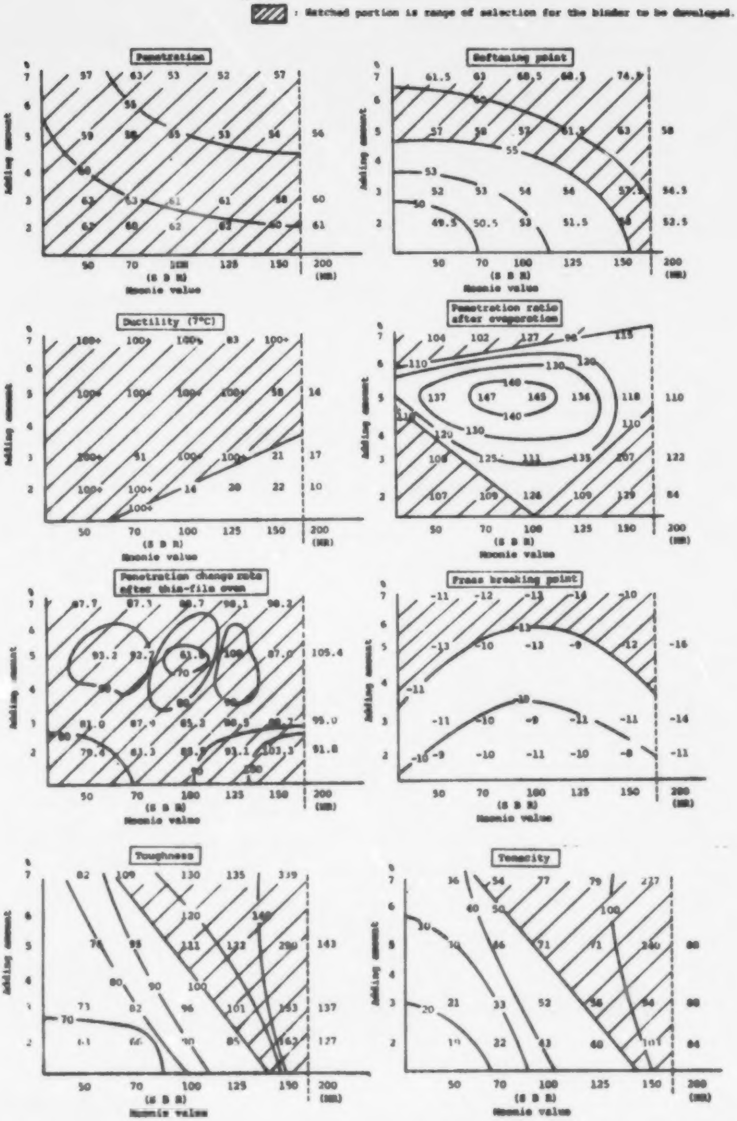


Fig. 2.4(1) Evaluation Against the Development Objective Value for Each Characteristic

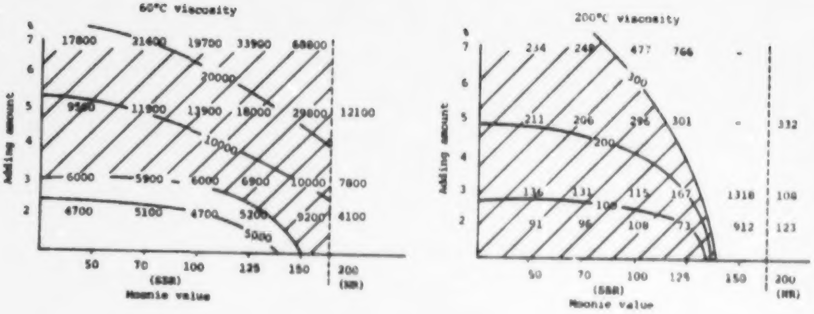


Fig. 2.4(2) Evaluation Against the Development Objective Value for Each Characteristic

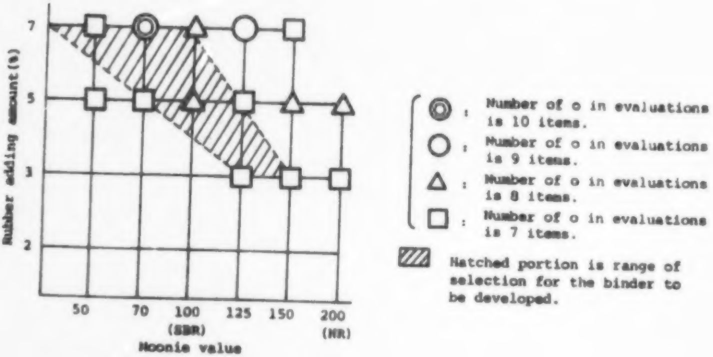


Fig. 2.5 Range of Selection for the Binder to be Developed

Paving
in Cold
Areas

Vancouver B.C., 1982

Session 6

Summary Report on the Study of Additives and Extenders for Asphalt Cements

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SUMMARY REPORT ON THE STUDY OF
ADDITIVES AND EXTENDERS FOR ASPHALT CEMENTS

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ABSTRACT

This paper summarizes a comprehensive study on asphalt additives and extenders in Canada. It was found that a number of major asphalt pavement problems continue to exist to varying degrees and they include the following:

- stripping,
- thermal cracking,
- rutting,
- hardening of binder,
- flushing,
- reflection cracking through overlays,

- segregation of aggregates,
- achievement of adequate binder consistency in recycled mixes, and
- high costs of bituminous materials.

In considering an additive or extender for any of these problems, the basic question should be asked, however, whether it will have any beneficial effects; if so will it also be cost effective? Or can the problem be equally well or better handled by using different materials, better construction practices or better control?

The causes of the problems identified can include the nature of the binder, nature of the aggregate, mix design, mix production and control, construction process, environmental factors, traffic factors and the structural design of the pavement.

The asphalt cement additives and extenders available in Canada can be categorized as follows:

- Anti-oxidants
- Anti-stripping agents
  - cationic surfactants
  - iron naphthenate
  - hydrated lime
  - organo silane
- Carbon black
- Foam suppressants
- Polymers
  - vulcanized rubber crumb
  - latex
  - solid polymers and rubbers
  - "chem-crete"
- Recycling agents
- Extenders - i.e., sulphur

These additives and extenders have been evaluated with respect to:

- cost-effectiveness,
- environmental effects,

- construction and performance effectiveness,
- energy effectiveness,
- availability, and
- recyclability.

The procedure developed for this evaluation can be used by any agency in Canada interested in evaluating an additive or extender for their particular conditions and requirements, and they can use it as a consistent set of "rules of the game" for suppliers and manufacturers.

The following additives and extenders have been shown to have promise or potential in solving the problems indicated:

- Stripping — anti-stripping agents
- Thermal cracking — sulphur with soft asphalts
- Rutting — sulphur-asphalt
- Hardening of the binder — anti-oxidants
- Flushing — none apparent
- Reflection cracking through overlays -- possibly rubber-asphalt interlayers
- Segregtion in the aggregate — none apparent
- Achieving adequate consistency of binder in recycled mixes — recycling agents
- High costs of bituminous materials — none at present time

However, the cost, energy and performance effectiveness of any of the foregoing additives and extenders, especially on a "life cycle" basis, has not been demonstrated.

A program of research is, therefore, recommended for the promising materials and this should involve both laboratory and field work, some of it regional and some if it on a nationally-coordinated basis.

1/ INTRODUCTION

1.1/ Background

A number of major asphalt pavement problems continue to exist in Canada. These have significant performance and economic implications. In 1981, the Council on Cooperative Research (CCR) of the Roads and Transportation Association of Canada (RTAC) considered that certain asphalt additives and extenders may offer some help to solving or alleviating these problems.

Consequently, a project on "Study of Asphalt Cement Additives and Extenders" was set up in 1981, and a contract awarded to the PMS Group of Paris, Ontario to carry out the work. This paper summarizes the results of their project report [1].

Project funding came from all the Provinces and Transport Canada. An RTAC project Committee monitored the work and provided direction and advice. Members of this Committee included Mr. W.A. Phang of Ontario as Chairman, Mr. W. Crawford of Gulf Canada Ltd., Mr. R. W. Culley of Saskatchewan, Mr. F.A. Gervais of Nova Scotia, Mr. I. Gilbert of Transport Canada, Dr. J. Node Keyser of Ecole Polytechnique in Montreal and Mr. B.P. Shields of the Alberta Research Council.

1.2/ Scope of the Paper

The introductory part of this paper briefly describes the project objectives, provides a working definition for additives, and states the justification for using these materials.

The paper then identifies the major asphalt pavement problems in Canada, the potential role of additives, the types available and their characteristics. An evaluation procedure follows and it is used to identify the most promising types of additives. The supply-demand situation is briefly considered, and finally a set of research needs is presented.

1.3/ Project Objectives

The area of asphalt additives and extenders is somewhat complex; a variety of products and suppliers are available, and the evidence of behavior and performance is scattered and inconclusive. For these reasons, prospective users in provincial, federal and municipal highway agencies face a rather difficult task in sorting out this subject area and assessing whether or not a particular additive or extender can be applied to their problem, and with what technical and economic effectiveness.

This situation was recognized in the project objectives which were defined as follows:

1/ to review the state-of-knowledge on asphalt cement additives and extenders;
2/ to identify the major asphalt pavement problems in Canada, and the potential role of additives;
3/ to identify the types, characteristics, availability and degree of usage of the various additives;
4/ to evaluate the additives in 3, and identify the most promising ones;
5/ to examine the supply-demand situation; and
6/ to recommend any research needs.

1.4/ Definition

The following working definition has been developed and used for the purposes of the study:

An asphalt cement additive is a material which would normally be added to and/or mixed with the asphalt before mix production, or during mix production, to improve the properties and/or performance of the resulting binder and/or the mix, or where an aged binder is involved, as in recycling, to improve or restore the original properties of the aged binder. An asphalt cement extender is an additive which replaces a part of the asphalt cement that would normally be used in the mix, and may additionally result in performance improvements or better economy.

This definition excludes materials which are intended to affect the structural nature of the mix (i.e., fillers, reinforcement, aggregates, etc.) and liquid and emulsified asphalts.

1.5/ Why Use an Additive or Extender

The justification or reasons for using an additive or extender would include the following:
- to solve or alleviate a pavement problem; and
- to realize some benefits, i.e., economic, environmental, energy, application and performance.

These potential benefits are reflected in the evaluation procedure subsequently described in this paper.

2/ MAJOR ASPHALT PAVEMENT PROBLEMS IN CANADA AND THE POTENTIAL ROLE OF ADDITIVES

2.1/ The Basic Question

Before identifying the actual problems, there is a basic question which should be asked when considering any additive or extender:

Would an additive or extender have any beneficial effect? If so, would it be cost effective? Or, could the problem(s) be handled by using different materials, better control procedures, better construction practices, a different design, etc.?

In other words, an additive may or may not help a problem, but, in order to know this, we have to understand the problems and their causes, understand the alternative treatments, that might be available (including additives and extenders), and be able to effectively match problems with treatments.

2.2/ Major Asphalt Pavement Problems

A user-agency survey (primarily involving the Provinces) identified the problems listed in Table 1.

1/ Stripping was the most widely reported and is perhaps the most important problem in Canada and the U.S. today. It involves a displacement of asphalt film from the aggregate surface in the presence of water, and is highly complex with respect to causes, mechanisms, treatments and evaluation of the effectiveness of the treatments. Stripping can be manifested in several ways, including the following:

- surface ravelling,
- stripping of asphalt from aggregate at the bottom of the bituminous layer, and
- an unstable or "mushy" mix, due to a combination of stripped asphalt and water.

Stripping is an asphalt- and aggregate-specific problem. In addition, the amount and nature of the voids, amount of asphalt, amount and type of additive used, nature of the water phase (i.e., liquid, frozen, vapour), temperature and other factors can influence the degree and rate of stripping. Moreover, these factors combine or interact to varying degrees. Stripping has been the subject of much research, including that in Ontario [2, 3] and in the United States. (An extensive list of references is available in Reference 1, plus a more detailed discussion of the subject area.)

A large number of treatments are available for stripping, including over 100 anti-strip agents on the North American market. Evaluation of the effectiveness of these treatments must eventually be through field evidence. However, prior laboratory testing is a major part of the total evaluation process and a number of aggregate and mix tests exist for this purpose.

2/ Thermal cracking is recognised as one of the major asphalt pavement problems in both Canada and much of the United States. It has been quite widely investigated since the mid 1960s and an extensive amount of literature exists on the subject [4, 5].

The only "cure" with present methods and materials is complete reprocessing of the bituminous layer because the cracks very quickly reflect up through an overlay. However, as subsequently discussed, asphalt-rubber interlayers may have applicability in retarding or minimising reflection cracking. As well, sulphur-asphalt is known to be effective, in an indirect way, because it allows the use of much softer asphalts [6, 7].

3/ **Rutting** was reported as a significant problem by several agencies, particularly in the case of high traffic volume pavements. In many cases, however, the problem may be primarily related to mix or structural design. The binder itself could also be a factor, if it were too soft.

Thus, in attacking the problem of rutting, these factors would normally be considered, before an additive. If, however, a softer asphalt was required for low-temperature cracking resistance but was too low in stiffness for high temperature and traffic conditions even with a good mix design, then the use of an additive or extender such as sulphur could be of significant assistance.

4/ **Hardening** of the binder was not reported as a problem by all agencies but undoubtedly exists to varying degrees across Canada and the U.S. Similar to some of the other pavement problems, it may be more a function of mix design and construction, particularly in terms of initial overheating during production and/or subsequent high voids content in the mix, than to any particular characteristics of the binder.

Excessive hardening of the binder is manifested by brittleness of the mix and, possibly, cracking. It can also have an effect on recycling in that a higher percentage of virgin aggregate and/or a softer asphalt may have to be used in order to obtain a satisfactory consistency in the combined binder of the final mix.

Additives might be of assistance, provided that proper mix design and construction takes place. The type of additives that would likely be used would be anti-oxidants, as subsequently discussed.

5/ **Flushing** is another problem related to mix design and construction and, in some cases, consistency of the binder used in relation to temperature and traffic conditions. It apparently occurs most frequently under heavy traffic conditions as reported by several provinces.

Proper mix design and construction procedure plus selection of an appropriate binder may represent the best available solution. In the case of open friction courses, latex addition may be of use to control draining of the binder while in transport and flushing while in service.

6/ **Segregation** has been identified as a problem by several agencies and is, therefore, listed in Table 1 although it is not related to the asphalt itself. Rather, it seems to be a function of the mixing process, particularly where drum mixers are used.

7/ **Reflection cracking** through overlays is a major problem across Canada. It is basically an extension of the thermal cracking problem for asphalt pavements, or of the joints in a Portland cement concrete pavement.

One way to treat the problem is to reprocess the entire old and cracked bituminous layer. Quite often, however, this is not practical or economical, especially where partial cold milling of the existing surface is desired, or where ripping or otherwise lifting off the entire existing bituminous layer is uneconomical, impractical or results in too much contamination from the underlaying base or subgrade.

The use of a rubber-asphalt "stress absorbing membrane interlay" (SAMI) has been demonstrated to be very effective by some agencies, such as in Arizona [8], but little experience or evidence yet exists to show that equal or even partial effectiveness will be achieved in northern regions.

8/ **Consistency** of binders in recycled mixes, in terms of too low a penetration at 25°C and/or too high a viscosity at 60°C is a problem reported by several agencies in Canada. It can be handled by using a higher percentage of virgin aggregate so that a greater percentage of new, softer asphalt or rejuvenators can be used.

9/ **High costs of bituminous materials** (Table 1) is a self-explanatory problem. While it was directly reported by only one agency in the survey, it would undoubtedly be endorsed by almost all users.

2.3/ Matching the Major Problems to Possible Causes

The major problems of Category A in Table 1 can be matched to several possible causes, and this can form the basis for identifying potential treatment alternatives.

Table 2 provides this matching, using eight possible causes. While the causes are quite general, Table 2 can provide guidance in seeking and assessing potential treatment alternatives.

2.4/ Potential Treatment Alternatives

The causes of Table 2 themselves suggest treatment alternatives in the sense that, for example, if mix design can be a cause of rutting, then one should ensure that a mix is designed to not have excessive rutting for the traffic, environment and other factors which also influence the problem.

Regarding additives and extenders, we should also seek to match these as treatment alternatives to the major asphalt problems, while carefully recognizing (a) the possible causes and whether or not an additive could be useful, or whether the problem might be better controlled by, say, proper mix design, good construction control, etc., and (b) whether or not additives or extenders have been tried only to control the problem, have enjoyed widespread use and success, have not yet been successfully applied, etc., -- in other words, is there evidence to suggest an additive or extender could be of benefit?

Table 3 provides such an assessment. This assessment is subjective, and based on the opinions and experience of the authors.

3/ TYPES OF ADDITIVES AND EXTENDERS AVAILABLE IN CANADA AND DEGREE OF USAGE

A comprehensive survey of suppliers and user agencies was used to identify the various types, sources and locations, availability and costs, etc. of the various additives and extenders available in Canada. The details are contained in Reference 1. Also contained in this reference is a functional description of these materials (i.e., basic chemical composition or source) and the degree of usage by various agencies in Canada.

Regarding degree of usage, this is relatively minimal for all the additives and extenders. A primary reason is that clear evidence is not yet available to the user agencies as to the cost, energy and performance effectiveness of these materials for attacking their various problems. This indicates a clear need for a systematic program of Canadian laboratory and field research to establish effectiveness of the promising additives, and to develop procedures for future evaluation of any additives which may become available.

The major types of additives and extenders available and used (either on a trial basis, in research studies, or in some regular projects) in Canada were identified as follows:

- anti-oxidants (i.e., dialkyldithiocarbamate);
- anti-stripping agents
  - cationic surfactants,
  - iron naphthenate,
  - hydrated lime, and
  - silanes;
- carbon black;
- foam suppressants (ie., silicone);
- polymers
  - latex, such as neoprene or styrene-butadiene,
  - vulcanized tire rubber crumb,
  - dry rubber crumb from latex,
  - plastics, such as ground up mixtures of polyethylene and polypropylene, and
  - chem-crete;
- recycling agents (i.e., lube oil extracts of either high aromatic hydrocarbon content, or lower aromatic and higher saturated hydrocarbon content); and

- extenders (i.e., sulphur).

4/ EVALUATION OF ADDITIVES AND EXTENDERS

4.1/ Evaluation Approach

The point of view taken in developing the evaluation procedures is that of the user-agency engineer or technologist who is faced with one or more of the following:

- solving a problem where an additive or extender might help,
- evaluating an additive or extender proposed to the agency by a supplier or manufacturer,
- sorting out a range of additive or extender types under consideration by the agnecy to solve their problems, or
- finding out more about a particular material which has come to the agency's attention from other users, from reading the literature, etc.

With a systematic evaluation method, the engineer or technologist will be able to (a) be consistent in comparing additives and extenders, (b) be as fair as possible in assessing any particular product, and (c) directly show any supplier or manufacturer the way in which their product will be assessed; in fact, this can also work to the industry's advantage in that they can try to compile as much information as possible beforehand for the user agency to help in promoting and selling the product.

4.1.1/ Framework, Criteria and Method for Conducting Evaluation

A general framework for evaluation is shown in Figure 1, together with six major criteria (economic, environmental, physical, energy, availability and recyclability). Initiation of any evaluation effort would presumably be on the basis of one of the four questions at the top of the diagram.

The six criteria of Figure 1 are intended to relate directly to the major problems previously identified. For example, if the problem is stripping, then evaluation of any proposed or available additive not only as to whether it could solve the problem in physical terms (C. in Figure 1) but, also, as to the economic, environmental, energy, availability and recyclability implications (A., B. and D. to F. in Figure 1) will be necessary.

After obtaining as much information as possible for the six criteria, and their components, a summary evaluation (item G. in Figure 1) would then be done to (a) assess the relative "strength" of the information available, and (b) assess the additive or extender in overall terms, from very promising to not worthwile in its potential to solve a problem. For many products, there will likely be much unavailable information in the six criteria areas.

The six criteria have been used in developing a method for actually carrying out an evaluation of an additive or extender. It includes a detailed Evaluation Form which is contained, with examples, in Reference 1.

Th engineer or technologist faced with the need or responsibility to evaluate a product would "pass it through" this form. As well, if a marketing representative offers a new product, this form could be handed back with the explanation that these are the "rules of the game" used for evaluation and answers to as many of the questions as possible would be appreciated. This could be advantageous to both the user agency and the supplier in that it represents consistency and fairness in evaluation.

4.1.2/ Identification of the Most Promising Types

Application of the evaluation method to the available additives and extenders was carried out and the summary results are given in Table 4.

The assessment of an additive as promising in Table 4 cannot answer the question of whether it actually will, and to what degree, solve a particular user's problem in a particular set of circumstances. That must be determined by the user's own trials or experiments with the product, or from further research. Nevertheless, some guidance can be provided by matching the promising additives and extenders of Table 4 to the major problems to be solved (Table 1).

Table 5 provides this matching of problem areas and additives which appear promising. It utilizes the evaluations of Table 4. While Table 5 is relatively self explanatory, it might be particularly noted that the following additives or extenders appear most promising:

- anti-stripping agents (cationic surfactants, iron naphthenate, hydrated lime and silanes);
- rubber asphalt — for interlayers, to limit reflection cracking through overlays;
- sulphur asphalt — to reduce or minimize thermal cracking and rutting, and when the economics become favourable, to reduce the price of binder by use as an extender for asphalt; and
- recycling agents — to achieve adequate consistency of binder in recycled mixes.

Because of the promise of these additives, a detailed review on the state-of-knowlege on their characteristics, applications, etc. and the literature which exists is contained in Reference 1.

The evaluation of the additive or extender should also include an economic evaluation over the life cycle of the pavement, rather than on an initial-cost basis. While the required data is not available for any additive today, it is, nevertheless, the only way in which a proper and complete economic evaluation can be done. Reference 1 contains a description of the procedures for such an evaluation, including the factors to be considered and a hypothetical example.

5/ SUPPLY-DEMAND SITUATION FOR ADDITIVES AND EXTENDERS

The maximum potential demand for an additive is related to the tonnage of hot mix produced. But the actual demand may only be some small percentage of the maximum. This actual demand in a region could be calculated from:

(a) tonnage of hot mix in that area which would be affected by the problem for which the additive is used,
(b) the maximum potential tonnage of additive as a percent of (a), and
(c) the degree of penetration of the additive into (b).

The latter factor is difficult to estimate (which is why, perhaps, that good marketing studies are essential) and depends on the demonstrated effectiveness of the product, its actual cost and the incremental cost per ton of mix, the marketing effort and services provided, degree of ready availability, etc.

It was not possible in the study to rigorously evaluate supply-demand relationships, for several reasons:

- there was insufficient data available;
- the current volume of usage in Canada of most, if not all, of the additives and extenders considered in the study is very small;
- the volume of usage or demand will likely be quite sensitive to their cost, energy and performance effectiveness (this has not yet been demonstrated for most of the materials in this study); and
- there did not seem to be any current supply problem for any of the additives or extenders.

Consequently, it was decided to consider primarily the demand situation and to do this in terms of tonnage of hot mix used in Canada to indicate **maximum potential demand**. This can also be useful to manufacturers and suppliers in assessing the market potential and estimating penetration rates.

On the other hand, the consumers (provincial transport agencies, municipalities, commercial organizations, etc.) are primarily interested in whether adequate supplies will be available and at what price.

The user-agency surveys conducted in this study and in the concurrent RTAC hot-mix recycling project attempted to establish current hot-mix tonnages for the provinces and estimates of future amounts. Table 6 contains the aggregated results of the survey for 1981 to 1990. It may be noted that total volumes are expected to remain fairly constant.

Table 7 contains estimates of the maximum potential demand for the promising additives and extenders of the study, based on the hot-mix tonnage estimates of Table 6 and representative application rates.

Actual demand for all the products listed, at the current time, is only a very small proportion of the Table 7 estimates. In other words, the market penetration is still quite minimal. For example, the average annual potential for sulphur would be about 900,000 tonnes, but the actual amount currently used is under 1% of the potential. This is largely due to the current price situation, and the transportation costs from Western Canada to much of the Canadian market.

Actual demand estimates have not been made for the additives and extenders of Table 7 because, until their cost, performance and energy effectiveness is conclusively demonstrated, the demand volumes will likely continue to be very low in absolute and percent of market penetration terms.

It quickly became apparent in the conduct of this study that the supply side did not need any precise quantification because there appeared to be no basic supply problems at the current time or for the foreseeable future. The only possible exception might be sulphur and, consequently, an assessment of the supply of that product was carried out, as described in Reference 1.

Table 8 provides a qualitative assessment of the supply situation for the promising additives and extenders, based on survey responses from suppliers and user agencies, discussions with representatives of these agencies, etc.

RESEARCH NEEDS AND RECOMMENDATIONS

6.1/ Basic Considerations

When a promising additive or extender is identified, either from a structured type of evaluation previously described, from evidence elsewhere, or simply because it has been brought to someone's attention, the questions faced would include the following.

- What additional information should be acquired before anything is done?
- Should there only be some lab testing done to start with, before a decision is made to go ahead with field experiments?

- Should both laboratory experiments and a very modest field trial (involving perhaps only construction demonstration and very limited observations of behavior and performance) be concurrently started?
- Should laboratory experiments and a comprehensive set of statistically designed field trials be done?

An associated question, in light of the national nature of the research project, involves coordination of field and laboratory experiments between the various federal, provincial and municipal agencies, plus the industries involved. While such cooperation is certainly desirable in principle, it must have very clearly-defined objectives, a carefully laid-out research plan and it must address a problem of widespread concern. For example, a national cooperative effort involving all the provinces plus several federal agencies was carried out in the late 1950s and early 1960s and led to CGRA's well known "A Guide to the Structural Design of Flexible and Rigid Pavements in Canada" in 1965. A subsequent, similar effort led to RTAC's "Pavement Management Guide" in 1977.

A key to the success of such national efforts and cooperation is not only the will to cooperate but also experiments in the individual provinces or municipalities that are complementary to the others, use the same experimental procedures, and are well documented.

A very important point to remember is that while a research program such as that recommended in this Section may focus on evaluation of additives and extenders, the underlying thrust should be toward solving the problems listed in Table 1. The promising additives and extenders only represent a potential means, and not necessarily the only or complete one, for tackling the problems.

Figure 2 presents, in flow chart form, the basic elements involved in carrying out a research program. These are relatively self explanatory and illustrate that the first steps are a well-defined problem plus the development of a specific set of objectives. While such steps might appear obvious, there is all too much evidence of research projects where these were not followed.

6.2/ General Research Recommendations

This study has clearly illustrated that the current state of technology on all the asphalt additives and extenders available today does not tell us whether any of them are cost, performance or energy effective. However, there is sufficient evidence available, and the evaluations of this study have demonstrated that there are some which look promising enough that further research should be done. Consequently, the following general recommendations on such research can be offered.

1/ The following additives and extenders warrant at least some level of research effort:

- Anti-oxidants
- Anti-stripping agents
- Polymers
  - vulcanized rubber
  - latex
  - solid polymers and rubbers
- Recycling agents
- Sulphur

Table 9 summarizes this recommendation in terms of the initial research effort warranted, and whether this is appropriate on a national or regional basis only.

2/ Comprehensive research problem statements and plans should be developed for the recommended areas of Table 9. These should include specific statements of objectives, work tasks involved in the research, schedules, cost estimates, etc. A suggested format and extent of such problem statements would be that of the National Cooperative Highway Research Program in the United States. It is recommended that a small research project be commissioned to produce such a set of problem statements and plans, with priorities and estimated costs. As well, recommendations could be included as to whether the various projects should be done by public agencies, private agencies or, in some cases, combinations of both.

Expansion of the foregoing general recommendations, in specific terms, is contained in Reference 1.

7/ CONCLUSIONS

The major conclusions of this paper, and the study upon which it is based can be summarized as follows.

1/ A variety of major asphalt pavement problems continue to exist in Canada.

2/ A number of asphalt additives appear promising in alleviating or reducing these problems.

3/ A considerable amount of research and in-service observations of behavior is required, however, to establish the cost, energy and performance effectiveness of these additives.

ACKNOWLEDGEMENTS

A large number of people contributed to the study. They include representatives from additive suppliers, from user agencies, from research agencies and many individuals. It is, unfortunately, impossible to cite all of them but their cooperation and help is sincerely appreciated.

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Table 1/ Major Asphalt Problems Reported in Canada, and their Extent

A. Widespread Problems of Major Importance

Type of Problem	Extent
Stripping	occurs to varying degrees in all parts of Canada
Thermal cracking	widespread in areas of higher freezing index (Northern B.C., Prairie Provinces, N. Ontario, N. Quebec), but also occurs to some degree in other parts of Canada.
Rutting	occurs to varying degrees in all parts of Canada
Hardening of the binder	occurs in all parts of Canada; no apparent Regional effects
Flushing	occurs in all parts of Canada; no apparent Regional effects
Segregation in the aggregate	reported in various areas of Canada where drum mixers used; not a Regional or in-service perf. problem
Reflection cracking through overlays	occurs in all parts of Canada
Achieving adequate consistency of binder in recycled mixes	can occur in any part of Canada
High costs of bituminous materials*	

B. Important Problems But of a More Regional Nature

- Chip retention in surface treatments
- Skid resistance
- Development of surface treatment for high traffic roads
- Retained moisture in aggregate
- Development of good cold-mix techniques for secondary roads

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* Not a performance problem and only reported by one agency, but probably applicable, in general, to all agencies.

Table 2/ Major Asphalt Problems Matched to Possible Causes

Problem	Possible Causes								
	Type or Nature of Binder	Nature of Aggregate	Mix Design	Mix Produc. or Control	Constr. Processes or Control	Environ. or Regional Factors	Traffic Factors	Pvt. Struct. Design	Remarks
Stripping	✓	✓	✓	✓	✓	✓			
Thermal cracking	✓					✓			Aggregate and mix design could have minor influence
Rutting	✓	✓	✓		✓	✓	✓	✓	
Hardening of Binder	✓		✓	✓	✓	✓			
Flushing			✓	✓	✓	✓	✓		
Segregation in the Aggregate				✓	✓				
Reflection cracking through overlays						✓		✓	Binder and mix design could have influence (i.e., if SAMI used)
Achieving adequate consis. of binder in recycled mixes	✓		✓	✓					
High costs of bituminous materials	✓		✓						Primary "cause" is actual cost of bitumen

Table 3/ Major Asphalt Pavement Problems Matched to Potential Treatment Alternatives

A. Widespread Problems of Major Importance

Type of Problem	Have Additive/ Extenders Been Used on Problem	Could An Addit./Extend. Be of Benefit			Suitability of a Conventional Treatment Alternative
		Yes	No	Unknown	
Stripping	Yes (Anti-strip agents)	✓			Voids control can be very beneficial
Thermal cracking	Yes (Sulphur and soft asphalt)	✓			Can use softer or less temp. susc. binders
Rutting	Yes (Sulphur)	✓			Materials selection and mix design can solve problem
Hardening of binder	Yes (Anti-oxid. agents)	✓			Produc. and voids control can be very beneficial
Flushing	Not directly			✓	Mix design can have major effect
Segregation in the aggregate	No evidence found		✓		Plant design modif. may help
Reflection cracking through overlays	Yes (Rubber-asph. and fabrics)			✓ (for N. environ.)	No practical, efficient treatment known
Achieving adequate consis. of binder in recycled mixes	Yes (Rejuvenating Agents)	✓			Sufficient, soft asphalt and sufficient virgin aggregate can be used
High costs of bituminous materials	Yes (Sulphur)	✓			N.A.

B. Important Problems But of a More Regional Nature

					Remarks
Chip retention in surface treatments	Unknown			✓	
Skid resistance	Yes (large rubber crumb and verglimit)			✓	Additives used for icing problems
Development of surface treatment for high traffic roads	Unknown	✓			
Retained moisture in aggregate	Yes (Silicone)	✓			
Development of good cold-mix techniques for secondary roads	Unknown			✓	

Table 4/ Summary Results of the Evaluation

ADDITIVE	STRENGTH OF INFORMATION	GENERAL ASSESSMENT
	very strong — strong — medium — weak — very weak	promising and worth trying — promising; use on trial basis — lacking info, do lab tests — not worth while
Anti-oxidants	Weak (Results with antioxidants in asphalt by various researchers are inconsistent and unencouraging. While results with lead dialkylcarbamates in all trials appeared positive in one study (Ref. 34, Ch. 3), the effectiveness varied with the asphalt)	Promising, use on trial basis, be aware of boiling points, decomposition temperatures of various anti-oxidants being tested in hot-mix preparations.
Anti-stripping Agents	Strong (Several agents must be tested with the particular aggregate and asphalt to be used on a project in order to find that agent which acts as the most effective adhesion promoter for that particular job (i.e., agents appear to be asphalt and aggregate specific))	Promising, worth a try after lab tests indicate suitability of anti-stripping agent for particular aggregate and asphalt.
Carbon Black	Very weak (No published reports in Canada - one MTC trial carried out in 1978). U.S. reports suggest some reduction in temperature susceptibility; doubtful if extent of this reduction is sufficient for Canadian climates.	Probably not worthwhile in Canada.
Foam Suppressant	Very strong, DC-200 and APA-5 used regularly for this purpose.	Promising, worth a try if needed.
Polymers	Vulcanized Rubber: Strong (many published reports on use of vulcanized rubber-asphalt seal coats and interlayers in the U.S. Product being evaluated extensively in Saskatchewan in seal coat applications)	Promising, worth a try
	Medium (Introduction into hot mix fairly recent, effect not thoroughly evaluated).	Promising, use on trial basis.
	Latex: Strong to medium (Tenacity of the latex modified asphalt greatly improved at ambient conditions; properties under Canadian cold temperatures (-20 to -40 degrees C) not yet determined).	Promising, use on a trial basis.
	Solid Polymers & Rubbers: Medium (Proper dispersion often a problem, compatibility with asphalt must be considered; little information on performance at -20 to -40 degrees C)	Lacking information on Canadian applications, lab tests should be carried out to evaluate seal coat or mix properties.
	Chem-crete: Weak (Some published papers but most performance information is only anecdotal and/or not published)	Not worthwhile unless used with a soft asphalt; lab test for low-temperature cracking resistance should be done before considering any field trials.
Recycling Agents	Medium (Long term evaluations of performance of recycled pavements have yet to be published)	Promising, worth a try. Field trials using several agents in various sections of same road would be most instructive, but should be supported by prior lab tests.
Sulphur	Strong (Pavements are definitely stiffer at higher temperatures; this has not yet been shown to improve pavement life but low temperature cracking can be reduced because softer asphalts can be used, and rutting can be reduced; As a partial replacement for asphalt, sulphur has been shown to be quite effective; a number of sulphur-asphalt projects have been done as regular, full-scale contracts)	Promising, worth a try if and when economics are favourable; in the meantime, should consider building limited number of projects to gain long term performance information for region.

Table 5/ Most Promising Additives for Solving Asphalt Pavement Problems

PROBLEM	ADDITIVES	REMARKS
Stripping	Anti-stripping agents - cationic surfactants iron naphthenate, hydrated lime, silanes.	Several should be tested for each aggregate-asphalt combination prior to use in order to determine suitability for the area or site.
Thermal Cracking	Sulphur with soft asphalt	
Rutting	Sulfur-asphalt	Chem-Crete is a possibility but Canadian experience (B.C.) has shown extensive cracking. However, asphalt used was that which would normally be employed for job.
Hardening of Binder	Anti-oxidants	Anti-oxidants should be soluble in asphalt and stable at hot-mix temperatures; evidence as to effectiveness of anti-oxidants is weak.
Flushing	None	
Segregation in the Aggregate	None	
Reflection Cracking through Overlays	Possibly rubber asphalt interlayer	These materials have not yet been proven effective in northern climates; limited experience with fabric interlayers has not shown them to be effective in northern climates.
Achieving Adequate Consistency of Binder in Recycled Mix	Recycling agents	No controlled or longer term road tests have been done to recommend soft asphalts over aromatic oils. Viscosity (penetration) should not be the only criterion used in lab evaluation of recycling agents.
High Costs of Bituminous Materials	None	Sulphur extension of asphalt when economics become favourable.
Chip Retention in Surface Treatments	Latex rubber dispersed in hot asphalt or in asphalt emulsions	Not widely used, however tenacity of the latex rubber-asphalt bond to substrates strongly suggest the suitability of such an additive, accompanied, of course, by sufficient rolling to ensure wetting of chip by binder.
Skid Resistance	For deicing-particulate vulcanized rubber of 3.175 mm size	Alaska had very positive experience with rubber crumb; Verglimit was effective at temperatures as low as -8 degrees C in P.E.I. but ineffective in the much colder Manitoba winter climate.
Development of Surface Treatment for High Traffic Roads	Specialty products such as Actiflex, Styrelf and Rocopave (all containing polymers) are being promoted for this purpose	Products not evaluated in Canada; no published reports of evaluations in other countries (France, U.S.).
Retained Moisture in Aggregate	DC-200; APA-5 (dimethylpolysiloxane)	Prevents foaming and slumping caused by moisture presence, does not remove the moisture per se.
Development of Good Cold-Mix Techniques for Secondary Roads	Unknown	Additives would be a proprietary component in a suitable MS or SS emulsion, or MC or SC cutback; their effect would be to promote aggregate wetting and adhesion of binder to aggregate.

Table 6/ Aggregated Estimate of Hot-Mix Tonnage in Canada, Based on (Provincial) User-Agency Survey

Provincial (tonnes x 10^6) Average Annual for 1981-1990	Municipal and Private (tonnes x 10^6) Average Annual for 1981-1990	Total Asphalt Cement (tonnes x 10^6)* per year
15	15	1.5

* Calculated as 5% by weight of mix (i.e., $0.05 \times 30 \times 10^6 = 1.5 \times 10^6$)

Table 7/ Estimates of Maximum Potential Demand for Additives and Extenders in Canada

Additive	Representative Rate of Application	Total Potential Demand (tonnes per year)	Remarks
1. Antioxidants	?	?	very little experience or informatio
2. Anti-stripping Agents	0.1 to 1.0% by weight of asphalt (except for hydrated lime)	1,125 to 11,250	Does not include commercial paving
3. Foam Suppressants	1/5000 parts of asphalt	225	Does not include commercial paving
4. Polymers	3% by weight of mix	675,000	Does not include commercial paving
5. Sulphur	3% by weight of mix	900,000	Includes all hot mix
6. Recycling Agents	Assume 25% of all tonnage is recycle and 2.5% by weight of mix is recyle agent	187,500	

Table 8/ Qualitative Assessment of Current and Projected Supply-Side Situation for Additives and Extenders

Additive/Extender	Assessment
1. Anti-oxidants	Expected that suppliers will respond to any demand (which will likely be low for the next few years at least)
2. Anti-stripping Agents	Several types available and several suppliers; no supply problem foreseen and suppliers should respond to any increase in demand on part of user agencies
3. Foam suppressants	The products used are in good supply; no problems foreseen
4. Polymers	Discarded tires should continue to be in plentiful supply but rapid increase in demand could result in temporary shortage of rubber crumb in certain areas until manufacturers are able to respond. Latex supply should be able to respond to demand
5. Recycling agents	Basic source of lube oil stocks should represent no supply problems
6. Sulphur	Expected that supply of elemental sulphur should continue to be sufficient at least to 1990 (see Section 6.6)

Table 9/ General Recommendations on Initial Research Efforts for Promising Additives and Extenders

Additive/ Extender	Expected Benefits from Use of Additive	Initial Research Effort	National or Regional Effort
Anti-oxidants	Reduction of asphalt hardening	Lead dialkyldithiocarbamates appear promising; suggest program of lab research combined with small scale, pilot type of field tests if lab results are positive.	Could be done by several provinces who coordinate with each other and make results nationally available.
Anti-stripping agents	Prevent or retard asphalt stripping	Four agents (cationic surfactants, iron naphthenate, hydrated lime and organo silane) appear promising; suggest program of research which first identifies most appropriate test method(s), then comprehensive laboratory effort to evaluate effects of many variables, followed by field trials.	Problems of stripping are site materials and conditions specific - thus regional; but national program to develop unified, consistent method(s) of test and evaluati is recommended.
Polymers			
a) Vulcanized rubber crumb	Durable, aggregate retaining seal coats; crack propag. resistant interlayers	Field trials of interlayers are recommended.	Could be done by several provinces and municipalities who coordinate and make results nationally available.
b) Latex	Durable, aggregate retaining seal coats	Combined program of lab charac. and small-scale field trials seems worthwhile.	A few provinces and municipalities might initiate pilot field trials and make results nationally available.
c) Solid	Durable, aggregate retaining seal coats	Lab testing programs to evaluate properties and effects of variables.	"
Recycling Agents	Achieve proper consistency of binder in hot-mix recycling	Comprehensive laboratory program to evaluate physical and chemical properties of binder and characterize low-temperature, resilient modulus and other physical responses of recycled mixes.	National program; results should be applicable to all agencies.
Sulphur	i)asphalt extender ii)less rutting iii)less low-temp. cracking (with soft asphalt) iv)reduced pavement thickness(?)	Continuing field projects to obtain long term perf. record.	Current costs suggest only regional effort but future econ. and conditions may lead national benefits. Suggest all provinces consider limited amount of periodic sulphur-asphalt construction.

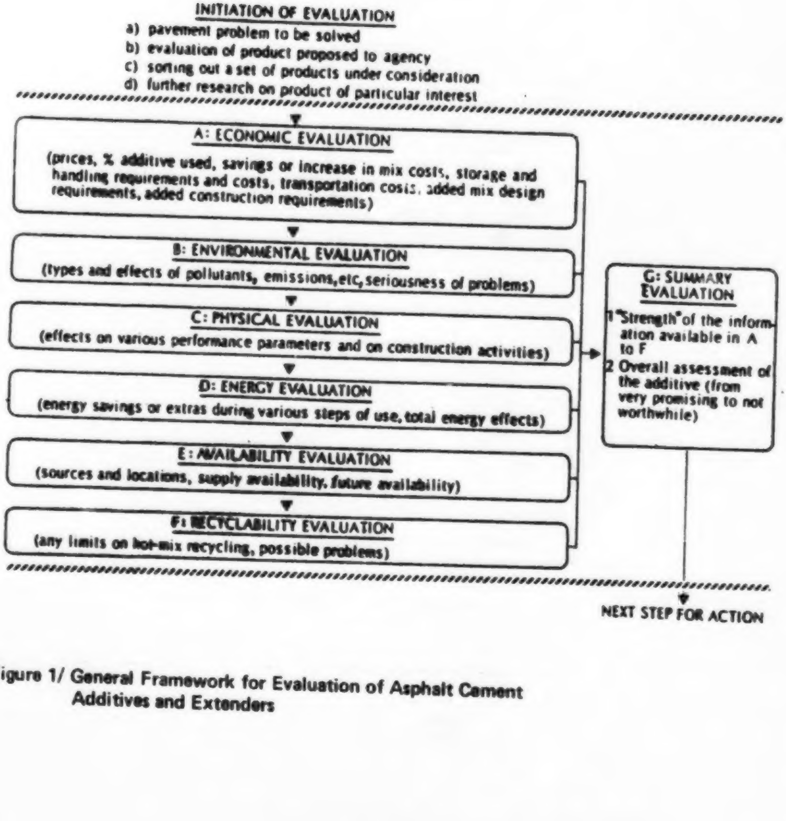
INITIATION OF EVALUATION
a) pavement problem to be solved
b) evaluation of product proposed to agency
c) sorting out a set of products under consideration
d) further research on product of particular interest
A: ECONOMIC EVALUATION
(prices, % additive used, savings or increase in mix costs, storage and handling requirements and costs, transportation costs, added mix design requirements, added construction requirements)
B: ENVIRONMENTAL EVALUATION
(types and effects of pollutants, emissions, etc, seriousness of problems)
C: PHYSICAL EVALUATION
(effects on various performance parameters and on construction activities)
D: ENERGY EVALUATION
(energy savings or extras during various steps of use, total energy effects)
E: AVAILABILITY EVALUATION
(sources and locations, supply availability, future availability)
F: RECYCLABILITY EVALUATION
(any limits on hot-mix recycling, possible problems)
G: SUMMARY EVALUATION
1 "Strength" of the information available in A to F
2 Overall assessment of the additive (from very promising to not worthwhile)
NEXT STEP FOR ACTION

Figure 1/ General Framework for Evaluation of Asphalt Cement Additives and Extenders

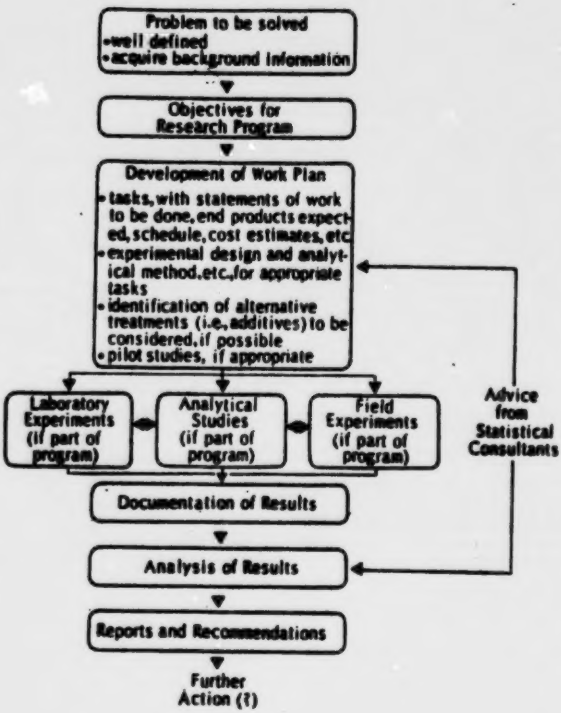
Problem to be solved
•well defined
•acquire background information
Objectives for Research Program
Development of Work Plan
•tasks, with statements of work to be done, end products expected, schedule, cost estimates, etc
•experimental design and analytical method, etc., for appropriate tasks
•identification of alternative treatments (i.e., additives) to be considered, if possible
•pilot studies, if appropriate
Laboratory Experiments (if part of program)
Analytical Studies (if part of program)
Field Experiments (if part of program)
Advice from Statistical Consultants
Documentation of Results
Analysis of Results
Reports and Recommendations
Further Action (?)

Figure 2/ Key Elements of a Research Program

Sponsor:

Transport Canada
Transportation
Development Centre

Supporting Agencies:

PUBLIC WORKS
RESEARCH INSTITUTE
MINISTRY OF CONSTRUCTION
JAPAN

Ministry of
Transportation and
Communications
Ontario